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Sediment Removal

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SEDIMENT REMOVAL

BY

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16. ABSTRACT

When properly conducted, sediment removal is an effective lake management technique. This chapter describes: 1) purposes of sediment removal, 2) environmental concerns, 3) appropriate depth of sediment removal, 4) sediment removal techniques 5) suitable lake conditions, 6) dredge selection and disposal area design, 7) exemplary case histories, and 8) costs. There are four major reasons for sediment removal from a lake: deepening, limitation of nutrient recycling, reduction of macrophyte nuisances, and the removal of sedimented toxic substances. Environmental concerns associated with lake sediment removal appear to be less negative than might be expected. Another potential impact is effect dredged material has on the area where it is disposed.

There is little doubt that dredging can be a successful lake restoration technique if there is a thorough pre-implementation evaluation of the lake setting, if proper equipment is selected, if the disposal areas are designed for end of treatment effectiveness and if the dredging is conducted by conscientious and competent operators. Successful examples of both surface sediment skimming and deep (6 to 8 m) dredging can be cited. Thorough pre-implementation evaluation is the most important aspect of the project.

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Sediment Removal

When properly conducted, sediment removal is an effective lake management technique. This chapter describes: 1) purposes of sediment removal, 2) environmental concerns, 3) appropriate depth of sediment removal, 4) sediment removal techniques, 5) suitable lake conditions, 6) dredge selection and disposal area design, 7) exemplary case histories, and 8) costs.

(1) Purposes of Sediment Removal

Deepening

When recreational activities are impaired due to shoaling, the only practical means of restoring lake use is to deepen it through sediment removal. Water volume of lakes must be sufficient to exceed water loss by seepage and evaporation which, according to the U.S. Department of Agriculture (1971), requires depths anywhere from 1.5 m to 4.5 m depending on the region of the country. Also, sufficient depth is required to avoid winterkill of fish. In colder parts of the United States lake depths must be at least 4.5 m (Toubier and Westmacott, 1976). These and other factors, such as intended lake use, availability of an area for disposal of material, and available funds must be considered in the design of a lake deepening project. The reasons for deepening and the means of measuring the success of such projects is the most straightforward of the sediment removal objectives. Since modern dredging equipment is reasonably efficient at moving large volumes of sediment, it is not surprising that all lake dredging projects undertaken for the purpose of deepening are considered to be successful (Pierce, 1970).

Nutrient Control

Shallow eutrophic lakes frequently are susceptible to periodic influxes of nutrients from the sediment. These inputs of phosphorus and nitrogen occur most often when a passing summer cold weather front depresses the thermocline (Stauffer and Lee, 1973). Thus, obnoxious algal blooms occur most frequently during peak recreation periods.

It has been estimated that sediment regenerated phosphorus amounted to approximately 45 percent of the phosphorus loading to Linsley Pond, Connecticut (Livingston and Boykin, 1962). Welch *et al*. (1979) estimated phosphorus inputs to Long Lake, Washington were 200-400 kg yr^{-1} or about 25-50 percent of the external loading. Shagawa Lake, Minnesota, experiences summer internal (from the sediment) phosphorus pulses of approximately 2000-3000 kg during June, July, and August. This compares to an annual phosphorus loading from the City of Ely, Minnesota of 5000-5500 kg before advanced waste treatment (AWT) and about 1000-1500 kg after AWT (Larsen *et al*., 1981). Before AWT sediment phosphorus loading to Shagawa Lake was about 28-35 percent of the total loading. Although total phosphorus loading to the lake decreased following AWT, the sediment input portion of the total loading increased to about 66 percent (Peterson, 1981). Sediment recycled phosphorus in Shagawa Lake has been sufficient to produce large summer algal blooms, thus slowing its predicted rate of recovery (Larsen *et al*., 1981).

The above indicates that phosphorus loading from sediment constitutes a significant portion of total phosphorus loading in some lakes. In cases where this can be documented, sediment removal might be expected to reduce the rate of internal nutrient recycling, thus improving overall water quality.

Toxic Substances

Toxic substances are becoming a common concern among industrialized nations. Insufficient treatment and disposal practices of the past have materialized into problems of today. Large scale surveys and improved analytical techniques are demonstrating that toxicants are more common to freshwater sediments than previously suspected (Horn and Hetling, 1978; Matsubara, 1979; Bremer, 1979). Many of the toxicants recycle from the sediment to the overlying water column where they may be bioaccumulated by aquatic organisms. Perhaps the most infamous incident of this type was the mercury pollution of Minimata Bay, Japan in 1956 (Fujiki and Tajima, 1973). More recent incidents in the United States have involved kepone contamination of the James River, Virginia

(Mackenthun et al., 1979) and PCB contamination of Waukegan Harbor, in the Great Lakes (Bremer et al., 1979).

The most obvious solution to the problem of contaminated sediment is removal. However, contaminated sediment removal frequently is complicated by secondary pollution of the overlying water column due to sediment agitation. Cutterhead hydraulic dredges have the ability to remove large volumes of sediment, but unfortunately also have the ability to resuspend large volumes of fine sediment (Suda, 1979; Barnard, 1978). Resuspension of sediment while dredging toxic substances must be minimized to prevent potential environmental damage. Therefore, proper selection of equipment becomes important to the success of toxic sediment removal.

Rooted Macrophyte Removal

An overabundance of rooted aquatic plants may interfere with fishing, boating, and swimming activities, and be aesthetically displeasing. Respiration by large plant masses in the littoral zone, during hours of darkness, may significantly reduce oxygen concentrations. In addition, there is an increasing literature on the effects of macrophytes on internal nutrient cycling. Their role in this process, with its effect on algal dynamics, may be an important reason for attempting to control macrophytes by removing them from a lake. Wetzel (1975) indicates that most of the organic matter found in small lakes may be derived from their littoral zone. Reimold (1972) showed that salt marsh plants

(*Spartina*) act as nutrient pumps, extracting sediment nutrients, translocating them to stems and leaves and excreting them to the surrounding water on each successive tidal cycle. Other researchers have documented that various species of freshwater aquatic plants extract nutrients chiefly from the sediment and translocate them to the surrounding water (Schults and Malueg, 1971; Twilley *et al*., 1977; Carignan and Kalff, 1980).

There is evidence that healthy freshwater aquatic macrophytes do not excrete large quantities of nutrients to the surrounding water while in the active growth phase (Barko and Smart, 1980). However, they do tend to concentrate sediment-supplied nutrients in their tissues. These nutrients are recycled to the lake when plants fruit, and during the senescence, death, and decay stages (Lie, 1979; Welch *et al*., 1979; Barko and Smart, 1979). Barko and Smart (1979) estimated that in-lake mobilization of phosphorus by *Myriophyllum* in Lake Wingra, Wisconsin, might amount to 62 percent of the annual external phosphorus loading. Welch *et al*. (1979) indicated that much of the "sediment" phosphorus loading in Long Lake, Washington, was probably due to rapid plant die-off and decay. Current information indicates that any long range lake restoration project concerned with in-lake nutrient controls will need to focus on both macrophytes and sediment (Carignan and Kalff, 1980; Barko, 1980). It should be noted that aquatic plant control, rather than erradication, is the most sound management approach.

(2) Environmental Concerns

Peterson (1979) pointed out a number of potential environmental problems associated with sediment removal. The problems generally can be classified according to in-lake and disposal area types.

In-Lake Problems

In-lake problems commonly center around the resuspension of sediment during its removal by dredging. One of the most common problems is the liberation of nutrients. Phosphorus is of particular concern due to its high concentration in interstitial waters of eutrophic lakes. Dredge agitation and wind action tend to move dredge-disturbed nutrient-laden sediment into the euphotic zone of the lake, creating the potential for algal blooms. Churchill *et al*. (1975) reported increased phosphorus concentration in Lake Herman, South Dakota, coincident with cutter-head hydraulic dredging, but they noted no increased algal production. This lack of algal increase presumably was due to the high turbidity level. Dunst (1980), on the other hand, found increased algal production in Lilly Lake, Wisconsin, when hydraulic dredging began, but it was short-lived and never posed a nuisance level problem. While nutrient enrichment due to dredging can become a problem, in most cases the effects are short-term and negligible relative to the long-term benefits. Hydraulic dredging might produce the additional problem of an unplanned drawdown if pumping rates are high.

Another, and potentially greater problem, associated with resuspended sediments is the liberation of toxic substances. Although small lake sediment removal projects, for the purpose of removing toxic substances, have been relatively uncommon, there is a growing concern (Matsubara, 1979; Sakakibara and Hayashi, 1979; Bremer, 1979; Spencer Engineering, 1981). Fine particles pose the major concern. Murakami and Takeishi (1977) have shown that up to 99.7 percent of the polychlorinated biphenyls (PCB) associated with sediment is attached to particles less than 74 µm in diameter. This could pose a particular problem for freshwater dredging projects where particle settling times are significantly greater than for marine waters. Therefore, added precautions should be taken where contaminated sediments are present. The precautions may include special dredges (see Section 4) and/or special disposal and treatment techniques (Matsubara, 1979; Barnard and Hand, 1978).

A relatively common concern about dredging among fisheries managers is the destruction of the benthic fish food organisms. If the lake basin is dredged completely, two to three years may be required to reestablish the benthic fauna (Carline and Brynildson, 1977). However, if portions of the bottom are left undredged the reestablishment may be almost immediate (Andersson et al., 1975; Collett et al., 1981) or within one to two years (Crumpton and Wilbur, 1974). In any case, the effect on benthic communities appears to be short-lived and generally tolerable relative to the longer term benefits derived. The same appears to be true for game fish populations (Carline and Brynildson, 1977; Spitler, 1973).

The above concerns are associated primarily with dredging as a sediment removal technique. Another technique for sediment removal involves lake drawdown (lowering the water level) to expose the littoral sediments, or in some cases (Born *et al*., 1973) the entire lake basin, followed by removal of sediment with earth-moving equipment. Drawdown accompanied by bulldozer operation probably is more destructive of the benthic community than dredging. It may also pose additional nuisance problems such as noise, dust, and truck traffic. Section four will address dredging techniques that minimize many of the above concerns.

The major non-lake impact of sediment removal concerns the area chosen for dredged material disposal. Urban area disposal site location has become more acute in the United States with the promulgation of Section 404 of Public Law 92-500, which prohibits the filling of any wetland area exceeding 4.0 ha (10 acres) without a Federal permit.

The flooding of wooded areas with dredged material should be avoided. Flooding kills trees, providing unsightly evidence of an improper disposal plan. Disposal areas may become attractive nuisances in the legal sense and can be extremely dangerous. It may be advisable to fence and post them for safety.

A frequently used disposal method in recent years employs diking in upland areas. A not uncommon problem with these sites is dike failure and flooding of adjacent areas (Calhoun, 1979). Groundwater contamination may become a problem with upland disposal if sand or gravel underlies the disposal area. Water wells in these areas could become contaminated, potentially adding liability to the sediment removal project. Upland dredged material disposal areas commonly are used for a variety of purposes once they have been closed and dewatered. However, dewatered and apparently dried disposal areas can be quite deceiving. Even those with strong surface crusting, deep cracking, and vegetation, can swallow earth moving equipment if excavation is attempted too early. Disposal areas covered to depths greater than one meter should be thoroughly tested for support capabilities before attempting to rework them with heavy equipment.

Another problem not uncommon to lake dredging is underdesign of disposal area capacity. Unfortunately, these failings usually become apparent only after the project is fully operational. The problem may be associated with the slow settling rate of suspended sediment in freshwater (Wechler and Cogley, 1977) and reduced ponding depth as the project proceeds. This may result in failure to meet suspended solids discharge permit requirements. If that happens there are two choices: 1) shut-down until seepage and evaporation allows additional filling, or 2) treat the discharge water. Either alternative adds additional cost to the project. Disposal area designs must be formulated for end of project efficiency, not average discharge requirements over the entire use period. Palermo _et al_. (1978) have summarized important technical information that will assist with the proper design, construction, and maintenance of dredged material disposal areas. The subject is also addressed in section six of this chapter. Barnard and

Hand (1978) describe when and how to treat disposal area discharges if standards cannot be met. Other reports by Brannon (1978), Chen *et al*. (1978), Gambrell *et al*. (1978) and Lunz *et al*. (1978) provide valuable information that will help minimize environmental problems at disposal sites.

(3) Sediment Removal Depth

When restoring a lake for sailing, power boating, and associated activities, the deepening requirements are relatively straightforward. However, when deepening to control internal nutrient cycling and macrophyte growth, the criteria are less clearly defined.

Lake Trummen, Sweden, is perhaps the most thoroughly documented case history of sediment removal to control internal nutrient cycling and macrophyte encroachment. Sediment removal depth in Lake Trummen was determined by mapping both the horizontal and vertical distribution of nutrients in the sediment. Digerfeldt (1971), as referenced by Bjork (1972), determined that approximately 40 cm of fine surface sediment accumulated from 1940 to 1965. Aerobic and anaerobic release rates of $PO_4^{\equiv}$-P and NH_4^+-N from this surface layer were markedly greater than for the underlying sediment. Based on these differences a plan was developed to remove the upper 40 cm of sediment. Results of the project are presented as a case history in section seven of this chapter.

Another approach to determining sediment removal depth has been proposed by Stefan and Hanson (1979) and by Stefan and Ford

(1975). This approach is similar to that developed by Stauffer and Lee (1973) which described thermocline erosion by wind in northern temperate lakes. Stefan and Hanson (1979), have used their model to predict the depth to which Hall Lake, Minnesota must be dredged to control adverse nutrient exchange from the sediment during the summer.

The Stefan and Hanson (1979) model assumes stable summer stratification is necessary to prevent enriched hypolimnetic waters from mixing into the epilimnion. Based on that assumption they calculated that Hall Lake (one of the Fairmont, Minnesota lakes) would require dredging to a maximum depth of 8.0 m. Dredging volume to obtain the 8.0 m depth would be enormous given Hall Lake's 2.25 km^2 surface area and 2.1 m mean depth.

There is little apparent chemical/physical distinction between shallow and deep sediments in Hall Lake. Phosphorus concentration is relatively uniform from the sediment surface to a depth of 8.5 m (737 to 1412 mg kg^{-1} for 37 samples, with a mean of 1097 mg kg^{-1}). However, it is possible that the phosphorus release rates from deeper sediment could be less than those of surface sediments (they were not measured). Furthermore, reduced nutrient release from the deeper sediment could be sufficient to significantly reduce the adverse impact of nutrients on the overlying water even though stratification might not be permanent (Bengtsson _et_ _al_., 1975). If that were the case, surface sediment skimming might produce nearly the same result as deep dredging and at a considerable saving. Therefore, it seems advisable to conduct incremental nutrient release rate experiments prior to adopting a lake temperature modeling approach to determine dredging depth for nutrient control.

Dredging will remove rooted macrophytes from the littoral zone of lakes, but there have been few detailed studies to determine depths necessary to prevent regrowth or nuisance plants. Factors influencing the areas in which rooted macrophytes grow include temperature, sediment texture, sediment nutrient content, and light level.

Using field data developed by Belonger (1969) and Modlin (1970), the Wisconsin Department of Natural Resources developed a guide to prescribe dredging depths necessary to control the regrowth of macrophytes. The guide was developed by plotting a regression between the maximum depth of plant growth in several Wisconsin lakes and their average summer water clarity, as measured by a Secchi disc. The relationship is described by the equation:

$$Y = 0.83 + 1.22X \qquad (8.1)$$

where,

Y = maximum depth of plant growth (meters)
X = average summer water clarity (meters)

According to Dunst (1980) the above relationship is used as a rough guide to develop dredging plans for macrophyte control. Dunst indicated, however, that dredging depths do not always need to exceed the predicted Y-value to achieve control. He indicated further that slight deepening frequently changes plant speciation to less objectionable forms.

Work by Collett et al. (1981) attempted to establish the depth of dredging necessary to prevent plant regrowth in the usually turbid Tuggerah Lakes of New South Wales. The strategy was to bracket the light compensation depth by dredging three 30 m^2 test plots 1.0, 1.4, and 1.8 m deep in a 30 by 180 m rectangular area parallel to and about 300 m from the lake shore. Three control plots of the same size (30 m^2) were undredged. Results indicated rapid recolonization (within 4 months) in the 1.0 m plots. One year after dredging, macrophyte biomass in the 1.0 m plots was about 60% of the predredging level. However, macrophytes had not reestablished in the 1.4 and 1.8 m test plots during the same year. Sediment nutrient levels were found to be similarly high in all test plots, hence nutrient deficiency was ruled out as a probable cause of reduced growth. The authors speculated that reduced light penetration at the 1.4 and 1.8 m depths may have limited regrowth, but also noted that deeper plots tended to fill with plant debris and lake detritus, thus altering the texture of the substrate. Unfortunately, no quantitative measurement of light level or sediment particle size was made.

The fact that macrophytes ordinarily grow to depths up to 2 m (Higginson, 1970) in the Tuggarah Lakes seems to imply that light alone should not have prevented regrowth at 1.4 and 1.8 m. The more flocculant sediment texture in deeper plots may have been a greater influence than was indicated by Collett et al. (1981). Their study did not answer conclusively the question about influence of light on regrowth of plants. It may even raise some question about the rationale for using light level to determine dredging depth. However, it seems to be a reasonable approach given what we know about macrophyte growth characteristics and *in situ* light requirements. The macrophyte depth/light level relationship is a very basic one. Hutchinson (1975) while describing the work of Maristo (1941) indicated that the maximum depth that autotrophic plants can grow is highly dependent upon the transparency of the water. The reliability of this relationship could be determined simply by plotting maximum depth of plant growth in summer for several individual lakes against average summer Secchi disc transparency and calculating the correlation coefficient.

Macrophyte control, whatever the method, must consider the need for fish spawning, waterfowl feeding, and general wildlife habitat requirements. Macrophyte control should never be equated with eradication. In Lake Trummen, Sweden, for example, about 30 percent of the lake area (a bay) was not dredged in order to provide a wildlife preserve.

(4) Sediment Removal Techniques

There are two major variations on sediment removal from freshwater lakes and reservoirs. The first one, involving lake drawdown followed by bulldozer and scraper excavation, has limited application. It has been used most successfully in small reservoirs (Born *et al.*, 1973). The obvious limitation of this technique is that water must be readily drained or pumped from the basin. A second drawback is that the basin must be allowed to dewater sufficiently before earthmoving equipment can operate. In spite of these problems, plus the added concern of truck traffic to transport the removed sediment, this approach has been used with some success at Steinmetz Lake, New York (Snow _et_ _al_., 1980).

The second, and by far the most common, sediment removal technique involves the use of dredges. Huston (1970) has reviewed the many types that are in use today. However, this chapter will address only those commonly used in lake dredging and those having special features which afford them a means for minimizing many of the adverse dredging concerns mentioned in section two.

The most common means of classifying dredges is by dividing them into bucket (mechanical) and hydraulic types. A third category of "special purpose dredges" is included here to highlight a move toward low turbidity systems for dredging fine grained and toxic sediments.

Grab Bucket Dredges

The only bucket dredges commonly used in lake restoration are the grab types illustrated in Figure 8-1. Figure 8-1A shows a clamshell bucket dredge in operation. Figure 8-1B depicts a typical Sauerman grab bucket setup. A common limitation of all grab bucket dredges is that they must discharge in the immediate vicinity of the sediment removal area or into barges or trucks for transportation to the disposal area. They normally are limited to a reach of no more than 30 to 40 meters. A major disadvantage of the grab bucket in lake restoration is the rough, uneven bottom contours that they create. Production rates are relatively slow due to the time consuming bucket swing, drop, close, retrieve, lift, and dump operating cycle. Grab dredges commonly create very turbid water conditions due to bucket drag on the bottom, pulling free from the sediment, dragging an open bucket through the water column, bucket leakage once it clears the water surface, and the occasional, intentional overflow of receiving barges to increase their solids content. Yet another disadvantage is that many lake sediments are highly flocculant, which reduces the pickup capabilities of a grab bucket.

Grab bucket dredges do have at least two advantages over other types. These are mobility and maneuverability. They can be transported with ease from one location to another and they can work in relatively confined areas. Thus, their chief use in lake restoration is shoreline modification, particularly around docks and marinas. They are not readily impeded by stumps and trash frequently found in these areas. A grab bucket operates most efficiently in nearshore areas containing soft to stiff muds. Depth is no impedence, but efficiency drops off rapidly with depth because of the time consuming operating cycle.

Some of the turbidity associated problems mentioned above can be overcome by using silt curtains. Silt curtains consist of a continuous polyethylene sheet (skirt) buoyed at the surface and weighted at the bottom so it hangs perpendicular to the water surface. It may, as shown in Figure 8.1, be used to encircle an

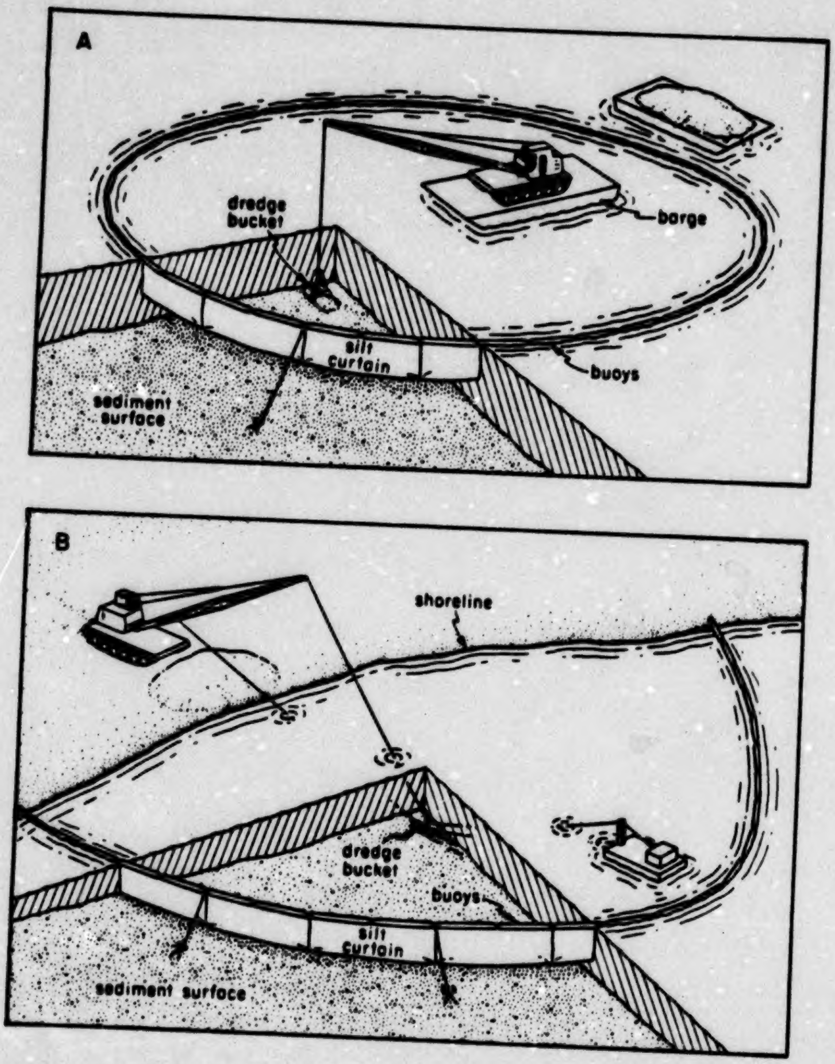

A
dredge bucket
barge
silt curtain
buoys
sediment surface
B
shoreline
dredge bucket
buoys
silt curtain
sediment surface

Figure 8.1. (A) Silt curtain encirclement of an open water grab dredge operation. (B) Shoreline isolation of a bucket dredge operation using a silt curtain.

open water dredging operation or isolate a length of shoreline where grab buckets are being used. The purpose of the silt curtain is to isolate turbidity conditions within the immediate dredging area, thus protecting clean surface water areas downwind. Silt curtains, while effective in controlling surface turbidity, are open at the bottom and will permit the escape of turbid water near the sediment water interface.

Another means of minimizing turbidity from grab bucket type dredging operations is to use a covered, watertight unit (Figure 8.2). Watertight buckets come in sizes ranging from 2 to 20 m^3. Manufacturers claim turbidity reductions from 30 to 70 percent compared to open buckets of comparable size. Dredged material production using watertight buckets is cleaner than conventional buckets, but still relatively inefficient compared to hydraulic dredges.

Hydraulic Dredges

There are many variations of hydraulic dredges including the suction dredge, the hopper, the dustpan, and the cutterhead section dredge. Hopper dredges are impractical for dredging small inland lakes. Cutterless suction dredges have not been used extensively. Attempts to use a suction dredge at Lilly Lake, Wisconsin, in 1978, were abandoned when it was discovered that the partially decomposed plant matrix in the sediment prevented it from "flowing" to the suction head (Dunst, 1982).

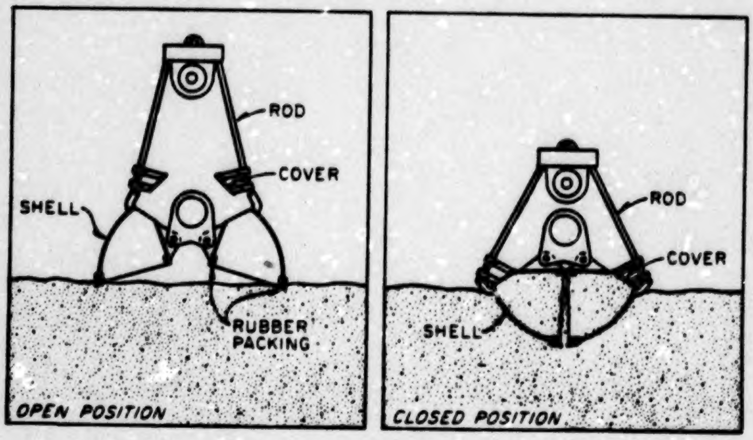

ROD
COVER
SHELL
RUBBER PACKING
OPEN POSITION
ROD
COVER
SHELL
CLOSED POSITION

Figure 8.2. Open and closed positions of the watertight bucket (redrawn from Barnard, 1978).

Dustpan dredges per se are not commonly employed in lake restoration. However, a "dustpan like" dredge was used to remove flocculant sediment from Green Lake, Washington, in 1961-62 (Pierce, 1970). The device consisted of a 15.25 m suction manifold with slot openings. The total size of the inlet ports was designed to produce inlet velocities of at least 300 cm sec^{-1}. As sediment consistency increased with depth, some of the inlet ports were sealed to increase flow velocity in the ones remaining open. The dustpan like suction head was barge mounted, designed to swing in a full 180 degree arc, and discharge into a 50.8 cm diameter pipeline. The discharge distance was about 792 meters. This dredge successfully removed 917,500 m^3 of sediment. Bjork (1974) indicated that the dredge head used on the Lake Trummen, Sweden, project had a specially designed "nozzle." The positive experience at Green Lake and at Lake Trummen indicates that dustpan types and other variations of conventional hydraulic suction heads should receive additional consideration for dredging highly flocculant freshwater lake sediments.

Inland lake sediment removal is most commonly accomplished with a cutterhead hydraulic pipeline dredge. This type of equipment has become the workhorse of the dredging industry and has become relatively common to inland lake dredging projects with the advent of small portable equipment. The primary components of any cutterhead dredge system include the: 1) hull, 2) cutter head, 3) ladder, 4) pump, 5) power unit, and 6) dredged material distribution pipeline (Figure 8.3).

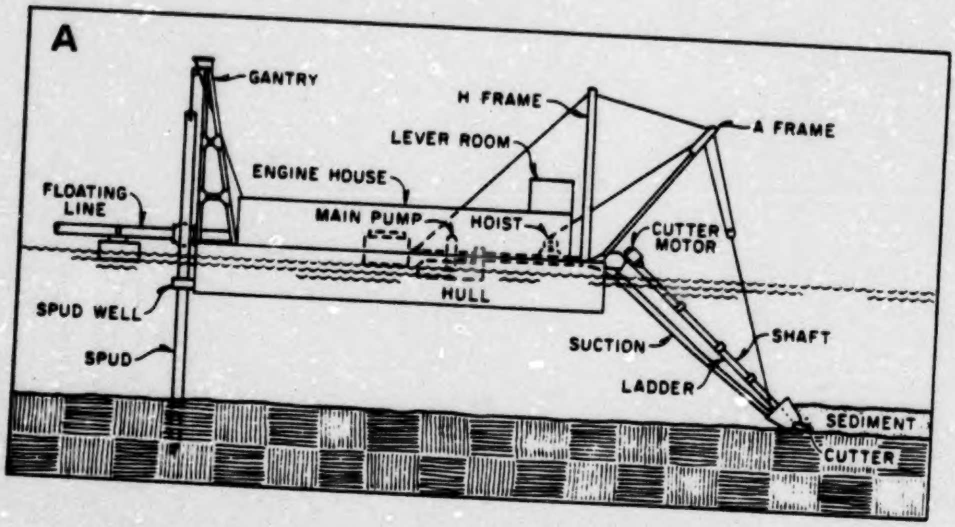

A
GANTRY
H FRAME
A FRAME
LEVER ROOM
ENGINE HOUSE
FLOATING LINE
MAIN PUMP
HOIST
CUTTER MOTOR
HULL
SPUD WELL
SPUD
SUCTION
SHAFT
LADDER
SEDIMENT
CUTTER

Figure 8.3. Configuration of a typical cutterhead dredge (from Barnard, 1978).

The dredge hull is made of steel construction designed to withstand the constant vibration created by the cutterhead when excavating consolidated material. The hull is the working platform which houses the main power plant, pump, lever room, and the assemblage of winches, wires, 'A' frames, etc. which comprise the dredge.

At the bow is a steel boom or ladder with a cutter mounted at its distal end. Ladder length determines the practical dredge depth limitations. The ladder also supports the suction pipe and the cutter drive motor and shaft. In some cases there may be a submersible auxillary suction pump mounted on the ladder. The ladder is raised and lowered by suspension cables attached at the outer end and to a hull mounted winch.

The cutter or cutterhead typically consists of three to six smooth or toothed conical shaped blades that rotate at 10 to 30 rpm to loosen compacted sediment (Bray, 1979). Cutter heads may be open nosed, closed nosed, straight vaned, ribbon screw shaped, or auger like. Most cutters have been designed specifically to loosen sand, silt, clay or even rock material. Few, if any, conventional hydraulic cutterheads have been designed to remove soft, flocculant lake sediment. Consequently, most of them probably are less efficient than they could be for lake dredging.

Spuds are vertically mounted pipes ranging from 25.4 to 127 cm in diameter, depending on the dredge size. The spuds are located at the stern of the dredge on both sides (Figure 8.4). They are used to "walk" the dredge forward by alternately raising and lowering them into the sediment.

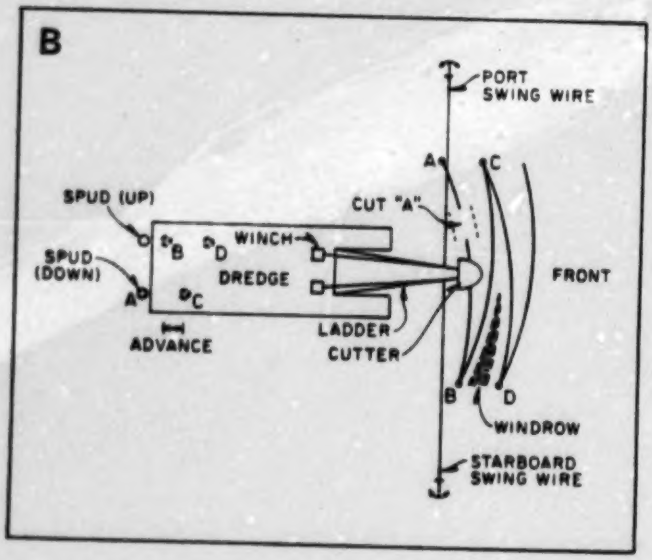

B
PORT SWING WIRE
A
C
CUT "A"
SPUD (UP)
B
D
WINCH
SPUD (DOWN)
DREDGE
FRONT
A
C
ADVANCE
LADDER
CUTTER
B
D
WINDROW
STARBOARD SWING WIRE

Figure 8.4. Spud stabbing method for forward movement and resultant pattern of cut (from Barnard, 1978).

Operationally, sediment loosened by the cutter moves to the pickup head by suction from the dredge pump, which usually is of the centrifugal type. The sediment slurry is then discharged by pipeline to a remote disposal area. Nomenclature of cutterhead dredges is described by the diameter of their discharge pipe. Hydraulic dredges used for inland lake work usually range in size from 15 to 35 cm, although the one used recently at Vancouver Lake, Washington, was 66 cm (Raymond and Cooper, 1984). Figure 8.4 shows how the cutterhead is moved from side to side and how the cut path is created by pulling alternately on the port and starboard swing wires. A major advantage of hydraulic cutter suction dredges over bucket types is that they are not confined in operation by limitation of cable reaches. Another advantage is their continuous operating cycle. This cycle permits hydraulic dredges to produce large volumes of dredged material. This ability, however, is not without its negative aspects. Most slurries from hydraulic dredges contain only 10 to 20 percent solids. Conversely, 80 to 90 percent of the slurry volume is water. This means that relatively large disposal areas with adequate residence times must be designed to precipitate solids from the dredge slurry. Also, it means that the large pumping capacity of hydraulic dredges might produce unplanned lake drawdowns unless disposal area overflow water is returned to the lake.

The amount of sediment supplied to the suction head is controlled by cutter rotation rate, thickness of the cut, and the swing rate (Barnard, 1978). Improper combination of any of these might cause the generation of excessive turbidity. Therefore, not only the configuration of the dredge equipment itself, but the skill of the dredge operator is important to minimizing turbidity.

Special Purpose Dredges

Portable cutterhead dredges are essentially miniatures of large coastal waterway dredges. The cutterheads of coastal dredges were designed for cutting sand, clay, and silt. They were not intended for use in fine, flocculant organic sediments (frequently 40 to 60 percent organics) encountered in lakes. Consequently, soft lake sediments have been a challenge to the dredging industry. Attempts to deal with the challenge has brought several dredging innovations. Among them is the cutterhead design used on Mud Cat® dredges. Mud Cat dredges utilize a horizontal auger to dislodge and move sediment to the center of the 2.4 m wide dredge head where it is sucked up by the pump and transported through the 20.3 cm discharge pipeline.

Figure 8.5 demonstrates the high mobility of the small Mud Cat dredges. Note also the mud shield which can be raised or lowered over the auger head to minimize resuspension of sediments. Nawrocki (1974) reported turbidity plumes due to dredging with a Mud Cat machine were confined to an area no more than 6 m distant from the dredge; however, operating conditions were not clearly defined. Suspended solids in the area of increased turbidity area ranged from 39 to 1260 mg liter^{-1}. Those near the bottom were approximately 1000 mg liter^{-1}. More turbidity is created by

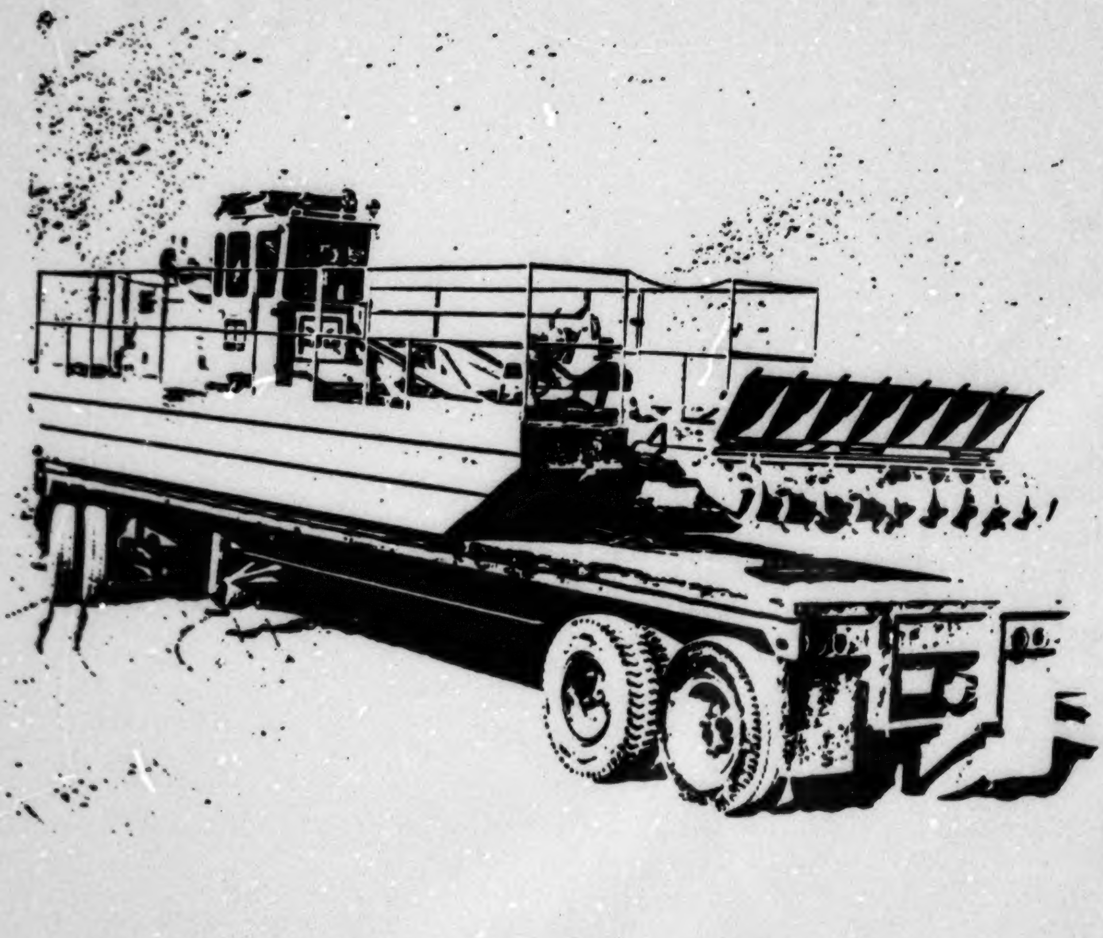


Figure 8.5. The Mud Cat dredge features a unique auger-type cutterhead. Size of the dredge makes it extremely portable (with permission of National Car Rental).

forward motion of the dredge than by backward motion. This appears to be associated with raising the mud shield while moving forward and lowering it when moving backward. Mallory and Nawrocki (1974) indicated the Mud Cat dredge should be capable of producing 30 to 40 percent solids. This represents nearly a doubling of the solids content commonly produced by conventional cutterhead dredges.

The Mud Cat guidance system is well suited to work on small water bodies. The dredge operates on a cable anchored at both shorelines. The guidance system permits uniform dredging of the bottom with few missed strips. Mud Cat dredges have been used successfully at Collins Park and several other small lakes in New York State. The portability, guidance system, reduced turbidity, and increased solids content resulting from use of these dredges appears to make them ideally suited to small lake restoration projects.

Clark (1983) reported on a survey of portable hydraulic dredges available for use in the United States. The survey identified 46 models of portable equipment available from several different manufacturers. No attempt was made to critically analyze the features of one dredge relative to another. However, tables are presented describing the general dredge specifications, the pump characteristics, suction and discharge diameters, cutter type, and working capacity. The information should be useful to engineers in selecting dredges since it includes dredging depth ranges from three to 18 m, production rate ranges from 15 to 1375 m^3 hr^{-1}, and a wide variety of cutterhead types.

Pneuma Dredging

There are a few other dredging systems which, although not commonly used for lake dredging, and in some cases currently unavailable in the United States, that deserve mention because of their unique capabilities. Each of these systems was developed specifically for soft, viscous sediment similar to that found in lakes. The machines were originally created on a large scale for harbor maintenance, however, it appears feasible to downsize them for lake dredging, which has been done in at least two instances (Matsubara, 1979; Spencer Engineering, 1981; Murakami, 1984).

The pneuma dredging system is unique among dredging systems. It was developed in Italy and employs air driven pumps. The system commonly consists of three cylindrical pump bodies, a pickup head, a compressor, and air lines to the pump bodies. This system operates on the principle that sediment and water, under hydrostatic pressure, will be forced into the pump body which is maintained at atmospheric pressure. Therefore, the filling efficiency increases with increased depth (increased hydrostatic pressure). At depths less than 10m, however, hydrostatic pressure is insufficient to operate the system effectively. The Japanese Oozer® dredge system (a modified pneuma design) has overcome the problem of operating at shallow depths by creating partial vacuum in the pump bodies during the filling phase. Partial vacuum in the pump bodies also permits more rapid filling at greater depths. Figure 8.6 is a schematic diagram of the Oozer dredge system. The use of vacuum and air pressure to respectively fill and empty

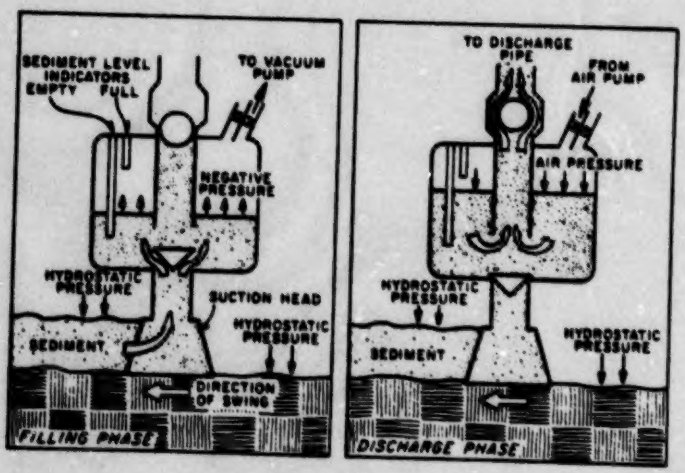

SEDIMENT LEVEL INDICATORS
EMPTY
FULL
TO VACUUM PUMP
NEGATIVE PRESSURE
HYDROSTATIC PRESSURE
SUCTION HEAD
HYDROSTATIC PRESSURE
SEDIMENT
DIRECTION OF SWING
FILLING PHASE
TO DISCHARGE PIPE
FROM AIR PUMP
AIR PRESSURE
HYDROSTATIC PRESSURE
HYDROSTATIC PRESSURE
SEDIMENT
DISCHARGE PHASE

Figure 8.6. Schematic of oozer dredge system.

the pump body and drive sediment through the discharge pipe, permits the removal of soft sediment at in situ density. This means the solid content of these dredge slurrys may run as high as 70 percent (Koba et al., 1975).

High solid content of the dredge material has several advantages, among them is that solids production rates are increased, sediment removal at in situ density minimizes its resuspension, and far less water volume must be contained at the disposal area.

Another dredge specialized for soft sediment removal is the Cleanup System® developed by TOA Harbor Works Company of Tokyo, Japan. This system employs a special suction head consisting of an auger, movable plates to direct sediment flow, a floating wing device to contain sediment as it is fed to the auger, and a large gas collection shroud (Figure 8.7). Methane and hydrogen sulfide gases, frequently liberated from organic sediment during dredging, are collected by the shroud thus preventing release of foul odors.

The structural configuration of the Oozer and Cleanup dredges are similar to a conventional cutterhead dredge (Figure 8.3). Likewise their cutpaths are created by swinging the cutter from port to starboard and vice-versa. The major differences are in the pickup head configuration, the cutter (auger) apparatus, and the pumps. As with most hydraulic cutterhead dredges, the Cleanup System has a large centrifugal pump mounted within the hull. However, in addition it has an auxillary submerged centrifugal pump mounted near the pickup head which facilitates dredging at depths greater than 9 meters. The combination of pickup head configuration and pump equipment, as with the Oozer System,

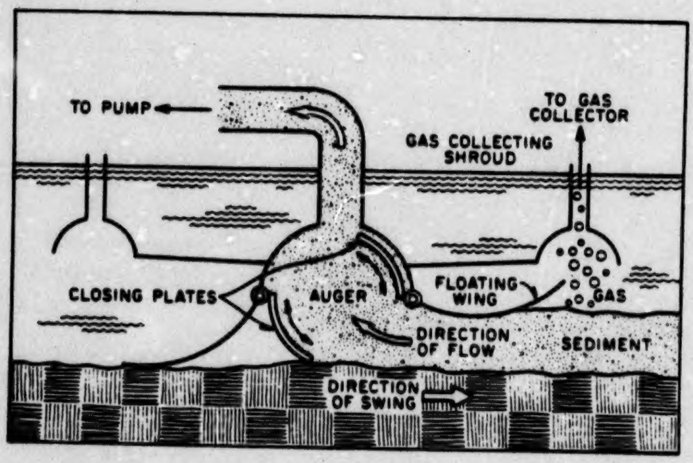

TO PUMP
TO GAS COLLECTOR
GAS COLLECTING SHROUD
CLOSING PLATES
AUGER
FLOATING WING
GAS
DIRECTION OF FLOW
SEDIMENT
DIRECTION OF SWING

Figure 8.7. Suction-head of cleanup dredge system.

permits the removal of soft sediment at in situ density (20 to 70 percent solids in the slurry) (Sato, 1978). Both systems were compared with conventional cutterhead equipment to determine their relative ability to minimize secondary pollution from sediment resuspension (Suda, 1979). Suspended solids concentrations 1.5 m above the sediment water interface, and directly over the two special dredge heads averaged 6 to 8 mg $liter^{-1}$. Under similar conditions the suspended solids concentrations around a conventional cutterhead averaged 40 to 80 mg $liter^{-1}$ at the same depth. Herbich and Brahme (1983), citing different references, reported average turbidity of 4.0 mg/l and 5.5 mg/l, three and seven meters above bottom for the clean-up and pneuma pump systems, respectively.

Although the clean-up and oozer dredges were developed by the Japanese to remove highly contaminated sediments from deep harbors, it demonstrates that extremely soft sediment can be removed efficiently with minimal resuspension if properly designed equipment is employed. A miniature prototype Oozer system has been used successfully to remove polluted sediment from Lake Kasumigaura, Japan (Matsubara, 1979). An improved version of the original dredge has been in use on the lake since 1978. Through 1981, it had removed 265,000 m^3 of sediment (Murakami, 1984).

A Pneuma Pump dredge was employed in 1981 at Gibraltar Lake, California where mercury contaminated sediment was removed. The Pneuma dredge was suspended from an "A" frame on a stationary barge (Spencer Engineering, 1981). The plan called for dropping the dredge into the sediment and letting it dig a cone of depression (pot holing method). However, as at Lilly Lake, Wisconsin, the plan failed when sediment did not flow.

A second method used was to tow the suspension barge so that the dredge was dragged over the sediment forcing it into the pickup head. The major problem with this approach was maintaining proper speed and depth on the pickup head. In spite of this, it was reported that the dredge pumped an average of 500-600 m^3 hr^{-1} at an average velocity of 2-3 m sec^{-1} and transported it 700 m horizontally and 52 m vertically to the disposal area. The suspended solids concentration of the slurry averaged 40 to 50 percent by volume and occasionally reached 70 percent. The slurry was air driven through the 35.5 cm discharge pipe by two compressors producing air at the rate of 40 m^3 min^{-1} and a third one producing 45 m^3 min^{-1}.

The Pneuma dredge performed well for a few months, after which it began to fail. Metal fatigue at the pump body seams was a major problem that eventually required rewelding all the seams. Another major problem was encountered when all of the rubber valve fittings were literally chewed up by debris. Both problems resulted in loss of air pressure and consequent loss of efficiency in the system. Replacement of rubber valves restored dredge efficiency, but the problem recurred in a few weeks. Others have reported similar problems concerning the rapid deterioration and plugging of Pneuma pump valves.

The contractor finally devised a valving system that used a combination of urethane and rubber (actually machined bowling balls) that was more durable than the original equipment. Spencer

Engineering (1981) reported this system to be much more satisfactory than original equipment and despite early problems the final engineering report indicated that the pump performed well compared to more conventional dredging methods and that it was economical and efficient in transporting toxic sediments without disturbance to water supplies or the environment (Spencer Engineering, 1981). Goldman _et_ _al_. (1981) confirmed the findings of Spencer Engineering. The chief concern in Gibraltar Lake was sedimented mercury. Goldman _et_ _al_. (1981) found that elevated levels of mercury were not detected in the water at any station or at any depth during dredging. In fact, the dredging was so clean that no bathing beach areas in the 110.8 ha lake were forced to close during any phase of the dredging. Thus, it appears that downsized and modified versions of the Pneuma system, and possibly other special purpose dredges may realize their greatest potential in lake restoration. Additional evaluation of these systems should be encouraged.

(5) Suitable Lake Conditions

When a lake has filled in sufficiently to be unusable for boating, swimming, and fishing activities, virtually nothing short of dredging will restore these uses. However, the high cost of dredging (Peterson, 1979) dictates that the feasibility of this treatment be examined closely in comparison to the intended use of the lake and alternative treatment methods (see Section 8 on costs).

Peterson (1981, 1982a) has described some of the problems in characterization of sediments that are unique to assessing the feasibility of dredging. Lake size might seem important to determining dredging feasibility. However, except for total cost, size appears not to be a constraint. Peterson's (1979) examination of 64 lake dredging projects showed that size ranged from less than 2 ha to over 1,050 ha and that volume of sediment removed ranged from a few hundred m^3 to over 7 million m^3.

The one aspect of large inland lake dredging that might become limiting is the requirement for a commensurately large disposal area. Restoration most frequently is sought for lakes in high use areas where sediment disposal space is scarce. It is also true, however, that the greatest user benefits will be derived from projects in densely populated areas (JACA, 1980). Therefore, it is important that disposal alternatives be explored for these situations.

Various productive uses of dredged material have been examined (Lunz _et al_., 1978; Walsh and Malkasian, 1978; and Spaine _et al_., 1978). A practical solution was employed at Nutting Lake, Massachusetts, where approximately 153 x 10^3 m^3 of sediment was sold as a soil conditioner at a price of \$1.40 m^{-3}. This reduced the total dredging cost by \$215,000 and the per unit dredging cost to about \$1.00 m^{-3} (Worth, 1981). The selling and removal of dredged material does not negate the need for a "disposal area," but it may reduce the size requirement of what actually becomes a sediment rehandling area. A seven ha, two-basin handling area with a total holding capacity of approximately 98 x 10^3 m^3 was used at Nutting Lake. This area provided sufficient capacity for a season's dredging. Before the next dredging season (summer), the material was removed and sold, thus restoring the initial disposal capacity. This indicates that lack of an adequate disposal area to accommodate all of the dredged material need not prevent a project from going forward.

Sediment removal projects should have reasonable assurance of longevity in order to be cost-effective. Therefore, an estimate of sedimentation rates should assist in determining the feasibility of dredging. Although dredging is expensive on a per unit basis of dredged material, when costs are amortized over the life expectancy of the project they may look much more reasonable. Sedimentation rates will help establish this "reasonableness." All other conditions being similar, lakes with relatively small watershed to surface ratios (nominally 10:1) will probably have lower sedimentation rates than those with large watersheds. Thus, the large lake/small watershed scenario should benefit more from dredging than will the reverse situation.

All of the above factors address physical features, i.e., depth, size, disposal, watershed area, and sedimentation rate. But, what of the chemical aspects of sediment and their influence on lake biota? Current information demonstrates that lakes with highly enriched surface sediments relative to underlying sediment might benefit from shallow dredging (Andersson et al., 1975; Bengtsson et al., 1975). Lake Trummen, Sweden showed marked changes in water chemistry and biota when 40 cm of rich surface sediments were removed (Bjork, 1978). Similar changes were observed in Steinmetz Lake, New York when 25 cm of organic sediments were removed and replaced by the same amount of clean sand (Snow et al., 1980). In both cases, extensive sediment surveys before dredging revealed that surface sediment was disproportionately rich in phosphorus and nitrogen relative to the deeper sediment. Open water sediment is usually more important in these surveys than littoral sediment since it becomes the sink. Littoral zones tend to be cleaned by wave action and, in the temperate zone, by spring ice scouring. The sediment survey should, at the minimum, determine the area of sediment to be removed and the depth. Horizontal sediment characteristics normally will be more uniform than vertical sediment profiles. Sediment depth may vary considerably depending on the basin configuration at the time of the lake formation or the transport of sediment to the lake via stream inlets. Vertical variation in the survey is important to note. Sediment profiles can be developed with the assistance of a Livingston piston corer. It will be important to note sediment color and texture differences with depth and to chemically characterize (phosphorus and nitrogen) the surface (0 to approximately 10 cm) sediment relative to deeper sediment if nutrient control is the intent (Peterson, 1981). Beyond this it will be quite useful to know the sediment particle size, settling rate, sediment volume, sediment bulking factor, etc. in order to properly select a dredge for the job and design an adequate disposal area.

Several variables may determine the suitability of a lake to dredging. However, generally the lakes most suitable will have shallow depths, low sedimentation rates, organically rich sediments, long hydraulic residence times, and the potential for extensive use following dredging.

(6) Dredge Selection and Disposal Area Design

Dredge Selection

Implementation of lake dredging requires several decisions over the course of a project. Among the most important ones are what dredging equipment to select and what factors to consider in the disposal area design. Equipment selection will depend on several variables including availability, project time constraints, slurry transport distances, discharge head, and the physical/chemical characteristics of the dredged material.

The primary factor controlling the design of disposal areas will be the amount of dredged material that must be contained. However, in concert with this requirement is the need to meet suspended solids discharge permit requirements. Therefore, factors such as sediment grain size, specific gravity, plasticity, and settling characteristics must be considered when designing the disposal area.

To illustrate the considerations most pertinent to dredge selection and disposal area design, the following examples are offered. A feasibility study has been conducted on hypothetical Dead Lake, located in a rural area of the glaciated upper midwestern United States. The study reveals the following lake characteristics:

Lake area = 120 ha
Maximum depth = 5.5 m
Average depth = 2.0 m
Normal water level = 245 m
Sediment water content = 30-60%

Additionally, since total project cost will most likely be calculated on the basis of sediment removal (payment is usually based on a measure of actual material removed), it was necessary to estimate the amount and type of sediment contained in the basin. The usual procedure for this is to collect hydrographic data suitable to developing a map showing the top of sediment depth and the configuration of the original basin, thereby describing the sediment volume. Accuracy of the sediment/ hydrographic map will depend on the sampling interval and the relief of the original basin. Even relatively shallow glaciated lakes may have "deep holes," thus reinforcing the need for sediment depth mapping.

Sediment sampling frequency for the purposes of determining volume will vary depending on the basin configuration and the desired accuracy of the survey. Preliminary sampling stations should be broadly spaced to provide a rough estimate of the solid bottom relief of the lake. This will help to define and hopefully limit the required number of stations for final mapping. Pierce

(1970) suggested that small to medium sized (< 40.5 ha) sediment removal projects should be mapped routinely by laying out sampling locations in a 15.25 m grid pattern. He suggested further that for lakes with surface areas greater than 40.5 ha, that the sample station grid size might be increased to 30.5 m without significant loss of accuracy. This would reduce the station frequency by half without adverse effects on mapping accuracy if the bottom relief was relatively uniform. Individual lake characteristics will ultimately dictate the required station frequency.

A common procedure for obtaining the necessary data is to make measurements at stations prescribed by the chosen grid size and relate the measurements to known elevation datum points on shore (topographic map, U.S. Geological Survey benchmark, etc.). The measurements can then be converted to elevations, thereby permitting the development of hydrographic maps and the calculation of sediment volume.

A simple means of obtaining the required data is to measure, at each station, the water depth to the sediment-water interface and the distance (depth) to which a probe can be pushed into the lake sediment before contacting hard bottom. Both measurements can be made at the same time by using a graduated probing ("sounding") rod. Lake sediment probes usually are steel rods measuring 0.95 to 1.6 cm in diameter. They can be bent. Therefore, an investigator needs to develop "a feel" for the degree of resistance which determines hard lake bottom. Sediment depth is determined by calculating the difference between the rod interval reading at "hard bottom" and the reading at the sediment-water interface. Distinction of the sediment/water interface may be difficult in lakes with highly organic, flocculent sediments. In these cases it is advisable to use a lightweight disc or foot at the tip of the probing rod to establish water depth to the sediment surface. Alternatives to this are the use of a graduated line and Secchi disc or an electronic depth sounder. It may be useful to employ divers to verify some of the depth determinations.

Depth determinations should be made during calm periods on open waters. A pontoon boat makes the operation much easier. In cold climates the work can be accomplished even more easily by making the measurements through holes drilled in the ice. Winter lake mapping makes it much easier to locate your position accurately and you need not content with a pitching boat. Pierce (1970) indicated that a properly equipped crew working efficiently should be able to collect water and sediment data in this manner over 10 to 20 acres of lake surface per day. The efficiency will be enhanced if data are collected during early winter before ice has thickened to more than 6 or 8 inches. Sediment depth measurements comprise an important aspect of the dredging project. Miscalculations in the sediment volume may lead to major errors in project cost estimates and to selection of improper dredging equipment. Therefore, accuracy should be stressed.

Sediment mapping on Dead Lake indicates that the deposits consist of highly organic silty material (muck). Water content of the sediment at the surface averages about 60 percent, while that at mid-depth and beyond ranges from 30 to 40 percent. Mapping data shows further that the thickness of sediment deposits are nearly 3.6 m at the south end, near the inlet and decrease in depth to approximately 1.8 m on the north end. These sediment conditions are well suited to the use of a hydraulic cutterhead dredge. Three sediment disposal areas have been located around the lake. The desire to minimize pumping distances thus makes it convenient to divide the lake surface area into three pieces and identify them with the nearest disposal areas. Figure 8.8 shows how the lake might be divided in order to best utilize the available upland disposal areas.

The feasibility study shows that sedimentation rates in the lake have been reduced significantly over the past 15 years as a result of shifts from row crop to small grain and hay crop farming in the watershed. The accumulated sediment is not contaminated and recent accumulations result mostly from autochthonous organic material decomposition. Therefore, it appears that deepending at least 15 percent of the lake to about 60 m, while leaving a fish spawning and wildlife area intact, will have a positive effect toward restoring the fishery and other beneficial uses. The study indicated further that water depth 60 m from shore should be a minimum of 2.5 m and that the bottom should then slope at a 5 percent grade to a maximum depth of 8.5 m. This depth would be highly desirable for maintaining adequate dissolved oxygen concentrations. The major portion of the lake should average about 3.5 m deep.

The maximum depth calculations based on the above recommendations indicate that approximately 1,530,000 m^3 of sediment will need to be removed. It is desirable to complete the project as rapidly as possible to minimize lake use disruption, so project duration is targeted for two years (encompassing mid-April through mid-November; ice free months, over two consecutive seasons).

Proper selection and use of hydraulic dredging equipment will permit implementation of the feasibility recommendations. Pierce (1970) suggested a series of considerations when selecting a hydraulic cutterhead dredge for any particular project. They are the following:

1. Plan to Optimize the Available Disposal Area

Long pumping distances to disposal areas should be minimized to the extent possible since energy requirements will increase with pumping distances. Examination of Figure 8.8 indicates that disposal area number one is the closest at 750 m when pumping from lake area number one. Area number two is 800 m and area number three is 1900 m when pumping from their respective lake areas. It has been calculated that areas one, two, and three will hold 574,000 m^3, 413,000 m^3, and 918,000 m^3 of dredged material, respectively. Therefore, area one would receive 564,000 m^3, area two would receive 413,000 m^3, and area three would receive the remainder of the total dredge volume (543,000 m^3) to optimize disposal efficiency by minimizing pipeline length. The lake configuration relative to the disposal locations indicates the shortest pipeline length will be about 300 m when working area

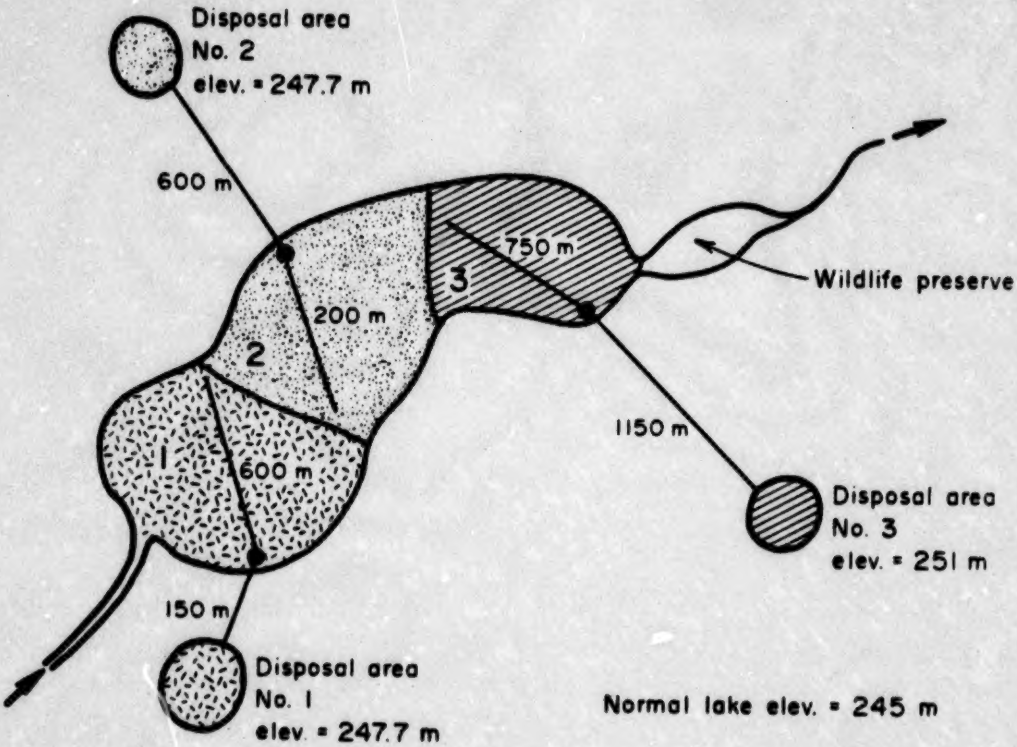

Disposal area
No. 2
elev. = 247.7 m
600 m
750 m
3
Wildlife preserve
200 m
2
1
600 m
1150 m
Disposal area
No. 3
elev. = 251 m
150 m
Disposal area
No. 1
elev. = 247.7 m
Normal lake elev. = 245 m

Figure 8.8. Dead Lake, showing the planned dredging areas, pipeline distances to disposal areas, and the wildlife area to remain undredged.

number one. It will be nearly the full 800 m in area number two. In area number three the maximum pipeline length will be 1900 m.

2. Analyze the Production Capacity of Available Dredging Equipment

It will be necessary to analyze the production capacity of various size dredges to determine which equipment might complete the job within the planned two-year period. A quick survey of equipment in the immediate area reveals that 20 cm, 25 cm, and 30 cm dredges are available. Therefore, production analysis will be limited to these sizes.

Dredge pump production rate ranges, rather than specific numbers, are usually listed by the manufacturers since dredging conditions and thus production varies considerably. Production ranges for the available dredges (20, 25, and 30 cm) are taken from Figure 8.9 to illustrate the method. Similar dredge capacity charts are available from various dredge pump manufacturers. Charts for the specific equipment in question should be used whenever they are available. From Figure 8.8 and the feasibility study, it is determined that the greatest sediment volume is located near the center of the lake and that transport from this area to the disposal cells will require pipeline transport distances in excess of 600 m. Therefore, the following dredge pump production range analysis was developed using the minimum, a medium, and the maximum pipeline lengths:

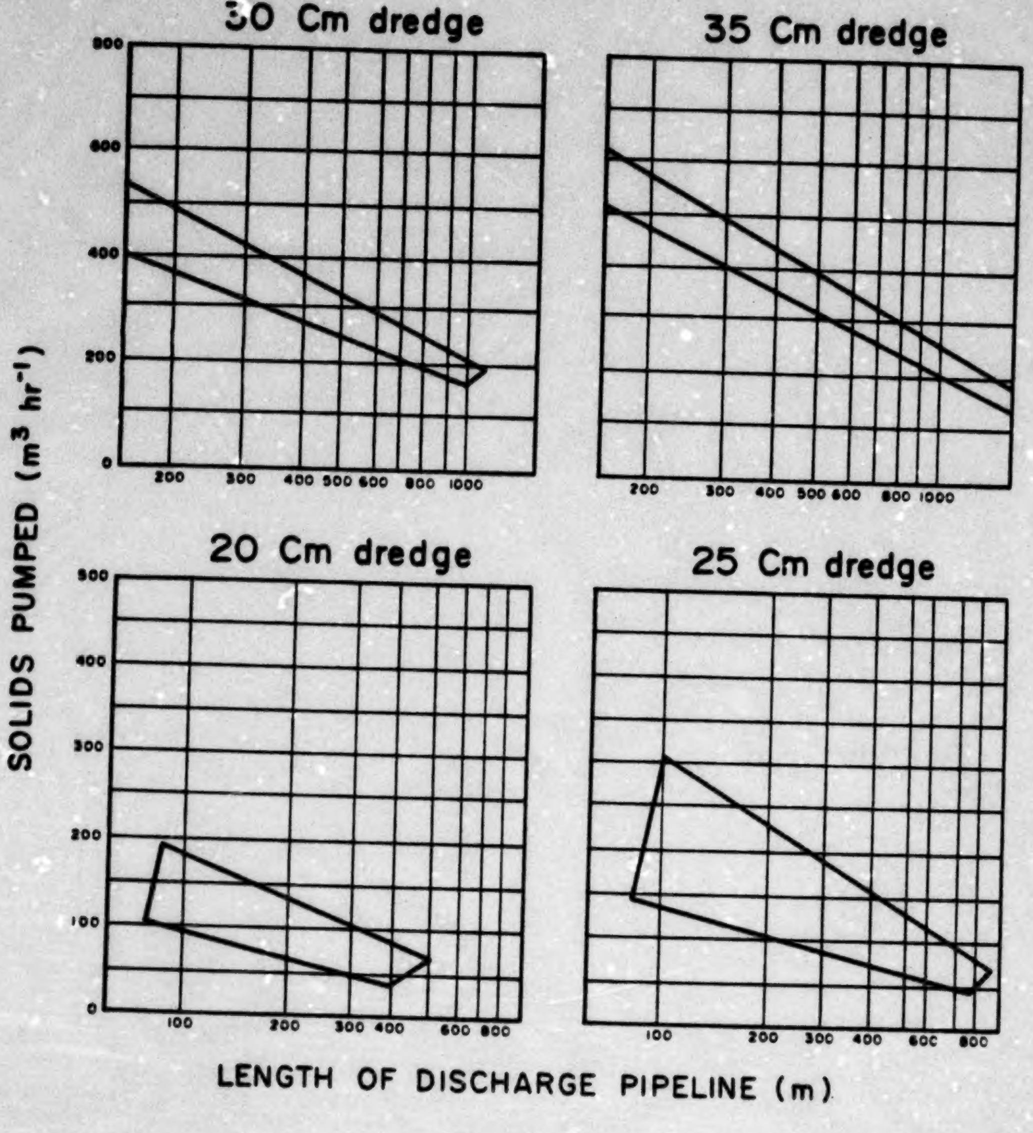


Figure 8.9. Dredged material production rate ranges for different sized dredge pumps (modified from Pierce, 1970).

300 meter length of discharge pipeline

20 cm pump = 50 to 110 m^3 per hour, average = 80

25 cm pump = 80 to 190 m^3 per hour, average = 135

30 cm pump = 310 to 420 m^3 per hour, average = 365

600 meter length of discharge pipeline

20 cm pump = beyond effective discharge length -- booster pump required

25 cm pump = 60 to 120 m^3 per hour, average = 90

30 cm pump = 220 to 290 m^3 per hour, average = 255

800 meter length of discharge pipeline

20 cm pump = beyond effective discharge length -- booster pump required

25 cm pump = 50 to 80 m^3 per hour, average = 65

30 cm pump = 190 to 250 m^3 per hour, average = 220

The analysis reveals that use of the 25 cm system for distances of 600 to 800 m becomes marginal in terms of efficiency. Also, it indicates that any disposal in cell number three from lake area number three, even with the largest system available (30 cm) will require the use of a booster pump.

3. Compute the Number of Dredging Days Required to Complete the Job

It was determined that approximately 1,530,000 m^3 of sediment must be removed from Dead Lake. A hydraulic dredge normally is operated 24 hours a day in three shifts for efficiency. There is always some down time for maintenance and pipeline relocation, so we can assume normal productive dredge time at approximately 20 hours per day, six days per week.

It was already observed from the above production analysis that the 20 cm dredge would not operate efficiently without a booster pump at discharge distances exceeding 600 meters. Since most of the pumping will require discharge at 600 meters and beyond, the 20 cm pump should not be given any further serious consideration. Using the average (rounded) discharge rates of the 25 and 30 cm systems for 600 and 800 m pipeline lengths we can calculate the number of days required to complete the Dead Lake dredging project.

25 cm Pump System

$$\frac{90 + 65}{2} = 77.5 \cong 75$$

$$\frac{1{,}530{,}000 \text{ m}^3}{(75 \text{ m}^3 \text{ hr}^{-1})\ (20 \text{ hr})} = 1020 \text{ days}$$

30 cm Pump System

$$\frac{255 + 220}{2} = 237.5 \cong 230$$

$$\frac{1{,}530{,}000\ \mathrm{m}^3}{(230\ \mathrm{m}^3\ \mathrm{hr}^{-1})\ (20\ \mathrm{hr})} = 237\ \mathrm{days}$$

Dead Lake is located in a north temperate zone of the United States which is subject to freezing conditions from approximately mid-November until mid-April. This reduces annual open-water work days to about 185. If the 25 cm pump system was used on this project, completion time would be in excess of five years (1020 ÷ 185 = 5.5 yr). If the project is to be completed within the targeted time of two years (two open-water seasons), the 30 cm dredge will be required (237 ÷ 185 = 1.3 yr). Disposal in area number three will require the use of an appropriately placed booster pump since the pipeline length to that area exceeds the efficient pumping distance (approximately 1000 m) of the 30 cm pump system.

4. Determine the Required Head-Discharge Characteristics of the Main Pump when Pumping Material with the Specific Gravity of Lake Sediment (approximately 1.20)

The required head discharge characteristics of a pump will be dependent on the discharge pipe length, i.e., the longer the pipeline, the higher the total head required. Pump head discharge characteristics must be analyzed for both minimum and maximum discharge distances. In the case of Dead Lake these distances are about 300 m when pumping from lake area one to disposal area one and about 1920 m when pumping from lake area three to disposal area three.

The sum of the total suction lift and total discharge head is defined by Pierce (1970) as the total dynamic head against which a pump must work. The heads commonly are computed from basic hydraulic formulas corrected for specific gravity of the pumped material. Suction lift incorporates suction elevation head, suction velocity head, and friction head in the suction pipe. The total discharge head is calculated by summing the pump velocity head, the discharge elevation head, and the friction head in the pipeline. Minor head losses usually are not corrected for.

Suction Head

Since the weight of dredged material (specific gravity of lake sediment is approximately 1.20) is greater than that of water, the surface of a column of water equal to the depth of Dead Lake would always be at greater elevation than the surface of an equal sized (diameter) column of dredged material of the same weight. The resultant difference in column heights is the suction elevation head. The suction elevation head is always referred to the horizontal center line of the main pump and is computed as follows:

$$h_{ss} = S_1A - S_2B \qquad (8.2)$$

where

h_{ss} = the suction elevation head (meters of fresh water)

S_1 = specific gravity of lake water (1.0)

S_2 = specifity gravity of material being pumped (1.2)

A = distance from the bottom of the cut to the water surface (m)

B = distance from the pump center to the bottom of the cut (m)

Assuming that the dredge pump is mounted on the hull at lake level and that maximum dredged depth will be 8.5 meters, the static suction head will be:

$$h_{ss} = 1.00\ (8.5) - 1.20\ (8.5)$$
$$h_{ss} = -1.7\ \text{m}$$

The minus sign indicates that a suction head exists. This number must be added positively to other heads computed for the suction system.

The suction velocity head is the energy required to start the movement of dredge material into the suction pipe. It can be computed as follows:

$$h_{sv} = S_2 \frac{Vs^2}{2\ g} \qquad (8.3)$$

where:

h_{sv} = velocity head (meters of fresh water)

S_2 = specific gravity of the material being pumped

V_s = velocity of the mixture in the suction pipe (m sec^{-1})

g = acceleration rate of gravity (m sec^{-2})

Normal suction pipe velocity should be maintained at 3.0 to 4.0 m sec^{-1} to assure that solids are carried into the pump. If we assume an upper-midrange suction pipe velocity of 3.6 m sec^{-1}, the velocity head in the suction pipe will be:

$$h_{sv} = (1.20)\,\frac{(3.6)^2}{2\,(9.82)}$$

$$h_{sv} = 0.8 \text{ m}$$

Friction losses caused by aqueous flow characteristics in pipes create the major portion of the head that a dredge pump must overcome. Pipeline friction loss is influenced by a variety of variables. Among them are the type of pipe, diameter of pipe, flow velocity in the pipe, the pipeline length, the pipeline configuration, and the percentage and type of solids in the pumped mixture. Since friction losses are magnified as the diameter of the suction pipe decreases, many small dredges utilize suction pipes one size (usually 5 cm increments) larger than the discharge pipe to reduce the losses. For example, a 30 cm dredge (size of discharge) might have a 35 cm suction line. Since the volume entering the larger suction pipe (35 cm), must be accommodated by the smaller discharge pipe (30 cm), the rate of velocity in the discharge pipe will be increased over that in the suction pipe. The velocity in the discharge pipe varies as the ratio of the square of the diameter of the larger pipe divided by the square of the diameter of the smaller one $[(35)^2 \div (30)^2 = 1.36]$. Therefore, the 3.6 m sec^{-1} velocity in the suction will be increased to approximately 4.9 sec^{-1} in the discharge. All influences affecting pipeline friction losses must be considered and applied to an acceptable equation for formulating friction losses. One of the most commonly used formulas is the Darcy-Weisbach pipe friction formula.

Pierce (1970) defines the suction friction head as the energy required to overcome friction losses in the pump suction line. The suction friction head can be computed using the Darcy-Weisbach formula:

$$h_{sf} = f\left[1 + \frac{(P-10)}{100}\right]\frac{LV_s^2}{2gD} \qquad (8.4)$$

where:

h_{sf} = friction head (meters of fresh water)
f = the friction factor
P = solids in dredge slurry (% by volume)
L = equivalent length of suction pipe (m)
V_s = velocity of the mixture in the suction pipe ($m\ sec^{-1}$)
g = acceleration rate of gravity ($m\ sec^{-2}$)
D = inside diameter of the suction pipe (m)

The friction factor (f) is a dimensionless number which is a function of the Reynolds Number and the relative roughness (absolute roughness ÷ diameter of pipe in m) of different types of pipe. The functions have been obtained experimentally and expressed graphically (Figure 8.10) by Moody (1944). The Reynolds Number can be calculated from the formula:

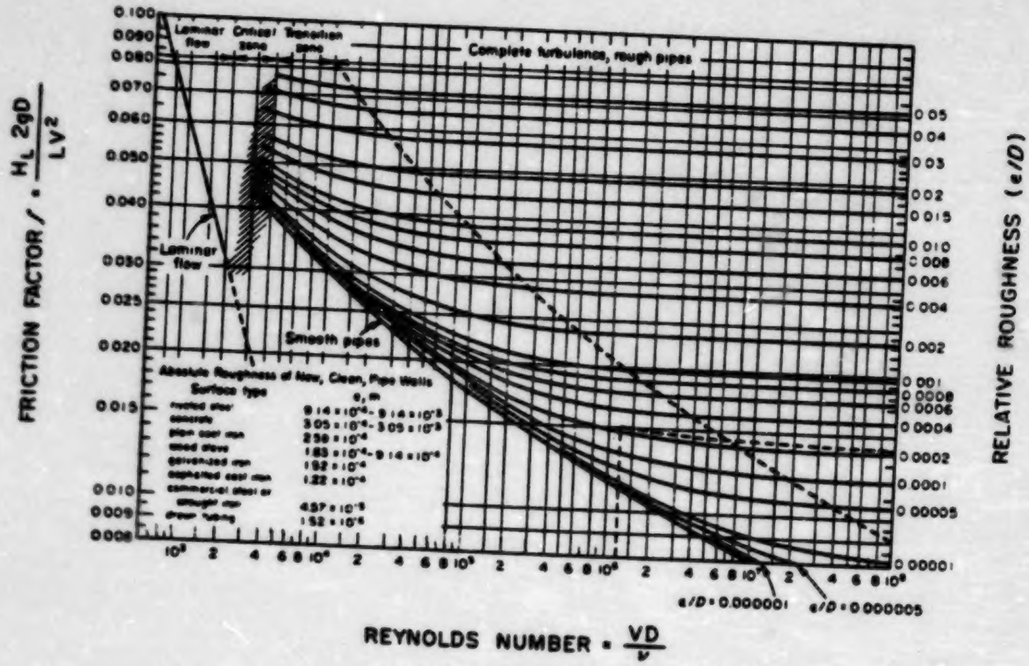


Figure 8.10. Moody diagram showing friction factors for pipe flows (redrawn from Randall, 1977).

$$R = \frac{VD}{v} \tag{8.5}$$

where:

V = velocity in the pipeline (m sec^{-1})

D = inside diameter of the pipeline (m)

v = temperature corrected kinematic viscosity of water (m^2 sec^{-1} x 10^{-6}), see Table 8.2

As stated above, the velocity of suction pipeline slurries commonly ranges from 3.0 to 4.0 m sec^{-1} or greater to maintain the suspension of solids (turbulent flow). If we use the previous assumed slurry velocity of 3.6 m sec^{-1} and a kinematic viscosity of water at 20°C (1.0 x 10^{-6} m^2 sec^{-1}) and apply these figures to a 35 cm (0.35 m) suction pipe, the following Reynolds Number can be calculated from Equation 8.5:

$$R = \frac{3.6\ (0.35)}{1.0 \times 10^{-6}}$$

$$R = 1.3 \times 10^{6}$$

If we assume a pipe roughness of 8.7 x 10^{-5} m (between commercial steel and drawn tubing) the relative roughness will be:

$$rr = \frac{\varepsilon}{D} \tag{8.6}$$

where:

rr = relative roughness

ε = absolute roughness (m)

D = inside diameter of pipe (m)

Table 8.2. Selected physical properites of water at various temperatures.

Temp T deg C	Density ρ (gm cm^{-3})	Viscosity μ (gm cm^{-1} sec^{-1} x 10^{-2})	Viscosity ν (cm^2 sec^{-1} x 10^{-2})*
0	0.9999	1.787	1.787
5	1.0000	1.514	1.514
10	0.9997	1.304	1.304
15	0.9991	1.137	1.138
20	0.9982	1.002	1.004
25	0.9971	0.891	0.894
30	0.9957	0.798	0.802
35	0.9941	0.720	0.725
40	0.9923	0.654	0.659
50	0.9881	0.548	0.554
60	0.9832	0.467	0.475
70	0.9778	0.405	0.414
80	0.9718	0.355	0.366
90	0.9653	0.316	0.327
100	0.9584	0.283	0.295

* cm^2 sec^{-1} x 10^{-4} = m^2 sec^{-1}.

$$rr = \frac{8.7 \times 10^{-5}}{0.35}$$

$$rr = 2.5 \times 10^{-4}$$

Applying relative roughness and the Reynolds Number to the Moody diagram (Figure 8.10), the resultant friction factor is 0.015 for these conditions. Note that the intersection of these two variables falls within the transition zone between laminar flow and complete turbulence in rough pipes.

Friction head losses in suction pipes vary with configuration, valving, suspended solids concentration, and cutterhead types. Cutterhead losses are highly variable and losses associated with fine grain dredge material is not well defined.

Correction factors for these variables are not readily available in tabular form. Therefore, engineering best judgment based on a combination of practical experience and laboratory tests frequently is applied to actual suction pipe lengths to calculate "the equivalent length of suction pipe." In effect, the equivalent length is a correction for suction pipe head loss. The suction pipe "correction factor" commonly is within the range of 1.3 to 1.7 (Hayes, personal communication). In order to dredge to a depth of 8.5 m (maximum lake depth after dredging), the dredge ladder (suction pipe length) will need to be approximately 15 m long. By applying a suction pipe equivalency correction factor of 1.7, the equivalent suction pipe length becomes 25.5 m (15 x 1.7 = 25.5). Substituting the required figures (assume 20% solids) into equation number 8.4 will result in a determination of the suction friction head.

$$h_{sf} = 0.015 \left[1+ \frac{(20-10)}{100}\right] \frac{25.5\ (3.6)^2}{2(9.82)\ (0.35)}$$

$$h_{sf} = 0.8 \text{ m}$$

The total suction head (H_s) on the dredge pump will be the sum of the suction elevation head, the velocity head, and the friction head.

$$H_s = h_{ss} = h_{sv} = h_{sf} \qquad (8.7)$$

$$= 1.7 + 0.8 + 0.8$$

$$H_s = 3.3 \text{ m}$$

Discharge Head

The discharge elevation head is represented by the difference in elevation (vertical distance) between the pump center line and the end of the discharge pipe corrected for the specific gravity of the dredge slurry. As mentioned previously, the specific gravity of lake sediment dredge slurry for this example is given as 1.20. The pump center line of dredges being considered for this job are at the water line of the dredge hull (from Figure 8.8, normal water level is 245 m). The top of the dike elevation at disposal sites one and two is determined to be 247.7 m. This information substituted into equation 8.8 will yield the discharge elvation head.

$$h_{de} = S_2 (E_D - E_p) \qquad (8.8)$$

where:

h_{de} = discharge elevation head (meters of fresh water)

S_2 = specific gravity of the mixture being pumped

E_D = elevation of the center line of the discharge pipe at the point of discharge (m)

E_p = elevation of the center line of the dredge pump (m)

Therefore, when pumping to disposal area numbers one and two, the discharge elevation head will be:

$$h_{de} = 1.20 (247.7 - 245.0)$$

$$h_{de} = 3.2 \text{ m}$$

The discharge friction head is the energy needed to overcome friction losses in the discharge pipe and it can be computed using the 8.4 equation form. The dredge pump will have to overcome maximum friction head when pumping from lake area two to disposal area two (greatest discharge distance without a booster pump). The pipeline length in this case will be about 200 m of floating pipe and 600 m of shore pipe. The two pipes differ considerably in joint configuration since the floating pipe must be flexible enough to accommodate wave action and relocation of the dredge. Therefore, the factor applied to the two types of pipe in order to calculate the equivalent length is different. Pierce (1970) indicates that the floating pipe factor typically ranges from 1.35 to 1.5 while that for shore pipe usually lies between 1.1 and 1.25. If we use the maximum factor of 1.5 for floating pipe (200 m) and the minimum of 1.1 for shore pipe (600 m) the factors will tend to normalize the pipeline equivalent lengths. Therefore:

Floating pipe length = 200(1.5) = 300
Shore pipe length = 600(1.1) = 660
Total equivalent length = 960 m

This total equivalent length substituted into the 8.4 equation form, together with the calculated discharge pipeline velocity (4.9 m sec^{-1}) will result in a discharge pipeline friction head loss of:

$$h_{df} = 0.015\left[1 + \frac{(20-10)}{100}\right] \frac{960(4.9)^2}{2(9.82)(0.30)}$$

$$h_{df} = 65.0 \text{ m}$$

The same value can be obtained from Figure 8.11 by entering the velocity scale at 4.9 m sec^{-1} and reading vertically to the 0.30 m pipeline intersection and then reading left to 2.05 on the friction head loss scale. The friction head loss scale is in meter per 30.5 m of pipe. Therefore, the scale reading must be multiplied by the number of times that 30.5 can be divided into the equivalent pipe length ($960 \div 30.5 = 31.47$, thus $2.05 \times 31.47 = 64.5 \cong 65$ m).

The pump velocity head is the energy required to increase the pump suction line velocity to the discharge pipeline velocity. It can be computed from equation 8.9.

$$h_{dv} = S_2 \frac{Vd^2 - Vs^2}{2(g)} \qquad (8.9)$$

where:

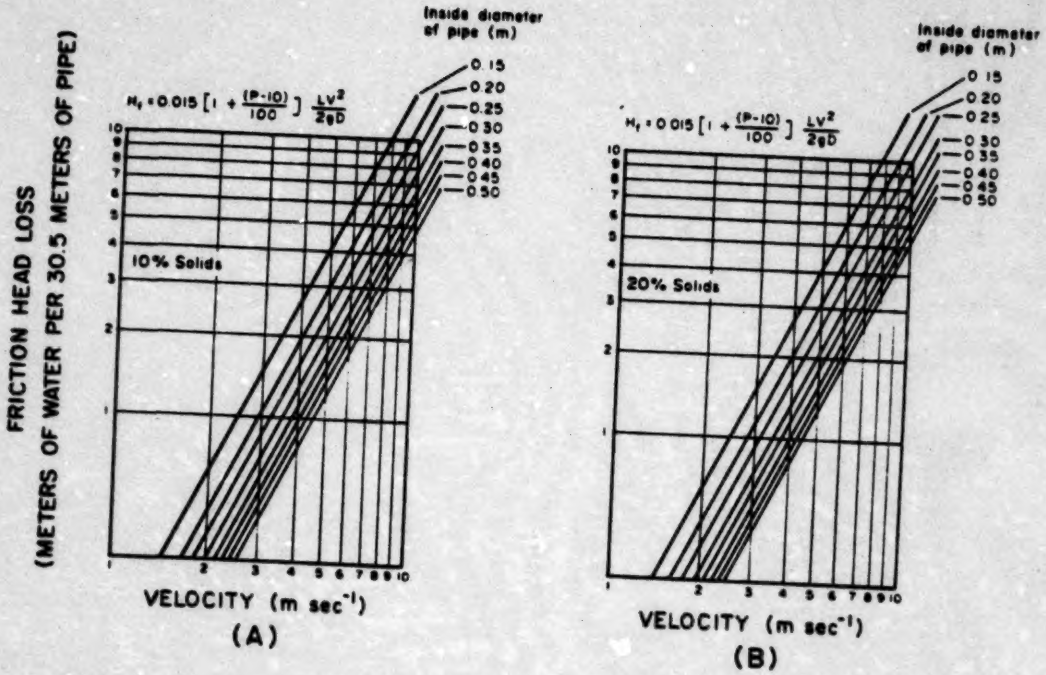
h_{dv} = pump velocity head (meters of fresh water)

S_2 = specific gravity of the dredged material

V_d = velocity of dredged material in the discharge pipeline (m sec^{-1})

V_s = velocity of dredged material in the suction pipeline (m sec^{-1})

The suction velocity for this example is 3.6 m sec^{-1} and the discharge velocity is 4.9 m sec^{-1}. Therefore the pump velocity head will be:



(A)

(B)

Figure 8.11. Friction head loss for 10 and 20 percent solids in various diameter pipelines as a function of slurry velocity (modified from Pierce, 1970).

$$h_{dv} = (1.20)\ \frac{(4.9)^2 - (3.6)^2}{2(9.82)}$$

$$h_{dv} = 0.7\ \text{m}$$

The total discharge head on the main pump is the sum of the discharge heads per equation 8.10.

$$H_d = h_{de} + h_{df} + h_{dv} \tag{8.10}$$

$$= 3.2 + 65.0 + 0.7$$

$$H_d = 68.9\ \text{m}$$

The total dynamic head on the main pump is the sum of the total suction head and the total discharge head per equation 8.11.

$$H_{TDH} = H_s + H_d \tag{8.11}$$

$$= 3.3 + 68.9$$

$$H_{TDH} = 72.2\ \text{m}$$

Once the total dynamic head is known, the power necessary to operate the pump against the resistances in the system can be calculated. However, it is also necessary to know the pump output. This can be calculated as follows:

$$Q = \frac{\pi}{4} = 3600\ D^2 V_d \tag{8.12}$$

where:

Q = output of the dredge pump ($m^3\ hr^{-1}$)

D = inside diameter of the discharge pipe (m)

V_d = velocity of slurry in the discharge pipe ($m\ sec^{-1}$)

The 30 cm dredge pump output when pipeline velocity is 4.9 m sec^{-1} will be:

$$Q = \frac{3.14}{4} \times 3600\ (0.30)^2\ (4.9)$$
$$Q = 1246\ m^3\ hr^{-1}$$

Therefore, a dredge pump should be selected that most nearly meets the required head discharge characteristics of 1248 m^3 hr^{-1} at a total dynamic head of 72.2 m. The performance curve for the dredge pump shown in Figure 8.12 meets these requirments at 'Point C.'

In addition, the dredge power plant must be sufficiently powerful to force the pump output through the pipeline. The required power is termed brake horsepower and can be computed from equation 8.13.

$$BHP = \frac{QH_{TDH}S_2}{2.737\ E}$$

where:

BHP = continuous brake horsepower at the pump

Q = dredge output (m^3 hr^{-1})

H_{TDH} = total dynamic head the pump works against (meters of fresh water)

S_2 = specific gravity of the dredged material

E = dredge pump efficiency (%)

Pierce (1970) notes that dredge pump efficiency of smaller dredges ranges from 50 to 65 percent, but that the efficiency

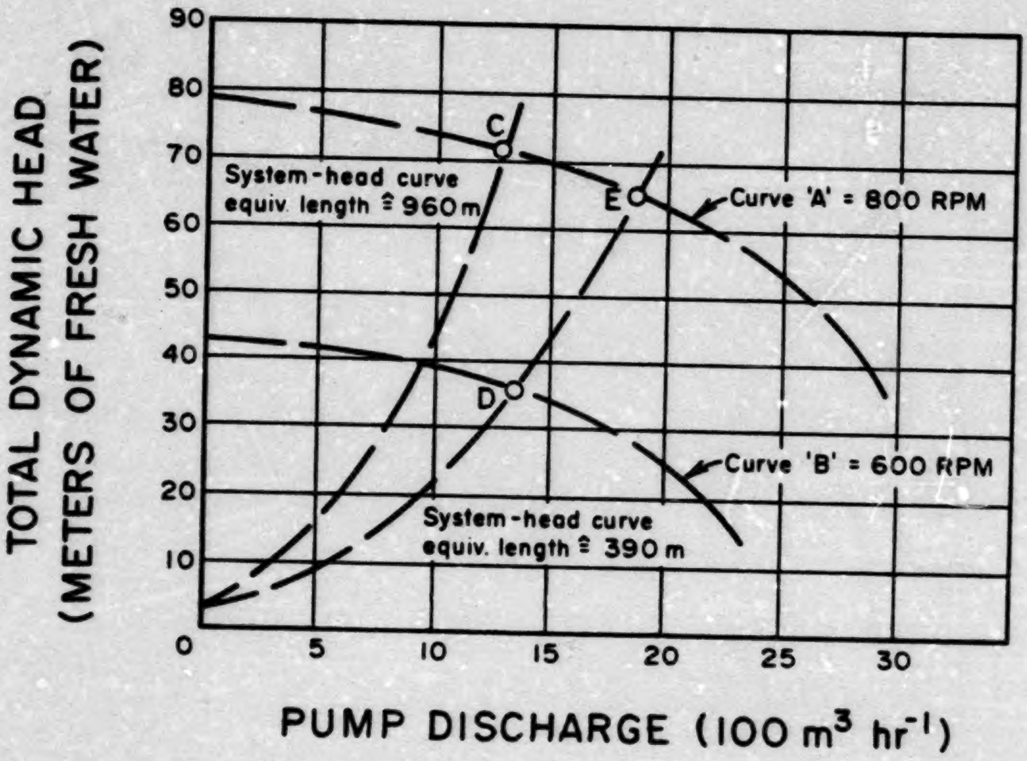


Figure 8.12. System head curve for a 30 cm dredge pump (modified from Pierce, 1970).

decreases with wear. Therefore, it is recommended that a conservative figure be adopted for efficiency. An efficiency figure of 55 percent is reasonable to assume. Therefore:

$$BHP = \frac{(1248)(72.2)(1.20)}{(2.737)(55.0)}$$

$$BHP = 718 \text{ hp}$$

An additional consideration for dredge power plants according to Pierce (1970) is that the manufacturers rated continuous duty capacity at any RPM should be discounted by at least 10 percent. This will assure that the power plant is adequately sized to rotate the pump at the required RPM (800 in this case) while at the same time it will provide a longer, more trouble free engine lifetime. In this case 718 x 1.10 = 790 hp, at 800 RPM, that will be required. Engine selection should be made based on a 1.5 to 1.0 reduction gear between the engine and the pump. Therefore, an engine of at least 790 horsepower at 1200 RPM should be used.

From the above dredge pump system analysis it can be concluded that:

1. The minimum dredge size that should be considered for this job is a 30 cm one with sufficient ladder length to dredge to a depth of 8.5 m.
2. The average production rate of a 30 cm dredge pumping distances of 600 m will be approximately 230 m^3 hr^{-1}.
3. The job can be completed in two summer seasons using a 30 cm dredge.
4. The closest disposal sites should be filled first.

5. There will be a maximum head of 72.2 m on the dredge pump when pumping from lake area two to disposal area two.
6. The dredge pump power plant should have a minimum continuous horsepower rating of 790 at 1200 RPM.

The above represents an analysis of maximum head conditions on the pump.

5. Determine Minimum Head Conditions When Pumping to the Nearest Disposal Area

The total head on the pump decreases as the pumping distance decreases. This means the pump output increases and the velocity of dredged material in the pipeline increases. As noted previously, a minimum pumping distance of about 300 m (150 m of shore pipe and 150 m of floating pipe) will be encountered when dredging and disposing in area one. By doing the same series of computations as were done for the 800 m pipeline the following can be concluded for the 300 m pipeline:

1. Average production rate for a 30 cm dredge with a 300 m discharge pipeline length is about 360 $m^3\ hr^{-1}$.
2. The system head curve for the 300 m pipeline is shown in Figure 8.12 as having an equivalent length of approximately 390 m. Point 'E' in Figure 8.12 shows that at 800 RPM the pump discharge would exceed 1800 $m^3\ hr^{-1}$. This would increase the discharge pipeline velocity beyond 6.5 $m\ sec^{-1}$. This would create excessive pump and pipeline wear, possibly

pump cavitation, and extreme taxing of the engine, all of which would create an inefficient operation.

3. There are two possible solutions to the problem. Install a smaller pump impeller or reduce the engine speed. Figure 8.12 shows that if the 30 cm pump is operated at 600 RPM in conjunction with a 390 m equivalent pipeline length (point 'D') that the pump will deliver about 1,340 m^3 hr^{-1} at a head of 36 m. According to Pierce (1970), at this capacity the discharge velocity would be reduced to an acceptable 5.1 m sec^{-1}.

4. The continuous horsepower required for the 36 m head and 600 RPM operating conditions are reduced to 385 hp. Using the 1.5 to 1 reduction would require the engine to operate at 900 RPM to turn the pump at 600 RPM.

From the above, it is apparent that the head discharge curves for a pump over its recommended speed range will be very helpful in making a dredge system selection and in determining the optimum pump speed at various discharge distances to maximize production. It should be noted that pump system performance will change as the system components wear through use. Therefore, it is a good policy to periodically check the performance against the manufacturer's rating curves. This is most easily accomplished by running tests on clear water when the system is first mobilized and rerunning the tests periodically after the pump has been in service. Results of these tests will permit the dredge operator to modify operating procedures as necessary to maintain optimum production.

6. Analyze Booster Pump Requirements for Pumping to Distances Beyond the Capacity of the Main Pump

The pipeline transport distance (1900 m) from lake area three to disposal site three (elevation = 251 m) exceeds the efficient capacity of the 30 cm dredge (see Figure 8.9) due to the increased friction head and reduced output from the pump. The booster pump selected must be capable of increasing the total discharge rate to maintain a minimum discharge pipeline velocity of 4.9 m sec^{-1}.

Figure 8.13 shows the head discharge for the main pump operating alone and the curve for the main pump and an identical booster pump operating in series. The "operating point" on the two pump curve can be determined by calculating the system head curve for the 1900 m discharge pipeline. For this system the equivalent length of the discharge line will be:

Floating pipe length = 760(1.5) = 1140
Shore pipe length = 1160(1.1) = 1276
Total equivalent length = 2416 m

The system head curve as shown in Figure 8.13 for the 1900 m discharge pipeline can be computed as follows:

$$h_{ss} = 1.0(8.5) - 1.20(8.5)$$

$$h_{ss} = -1.7 \text{ m}$$

$$h_{sv} = 1.2 \frac{(3.6)^2}{2(9.82)}$$

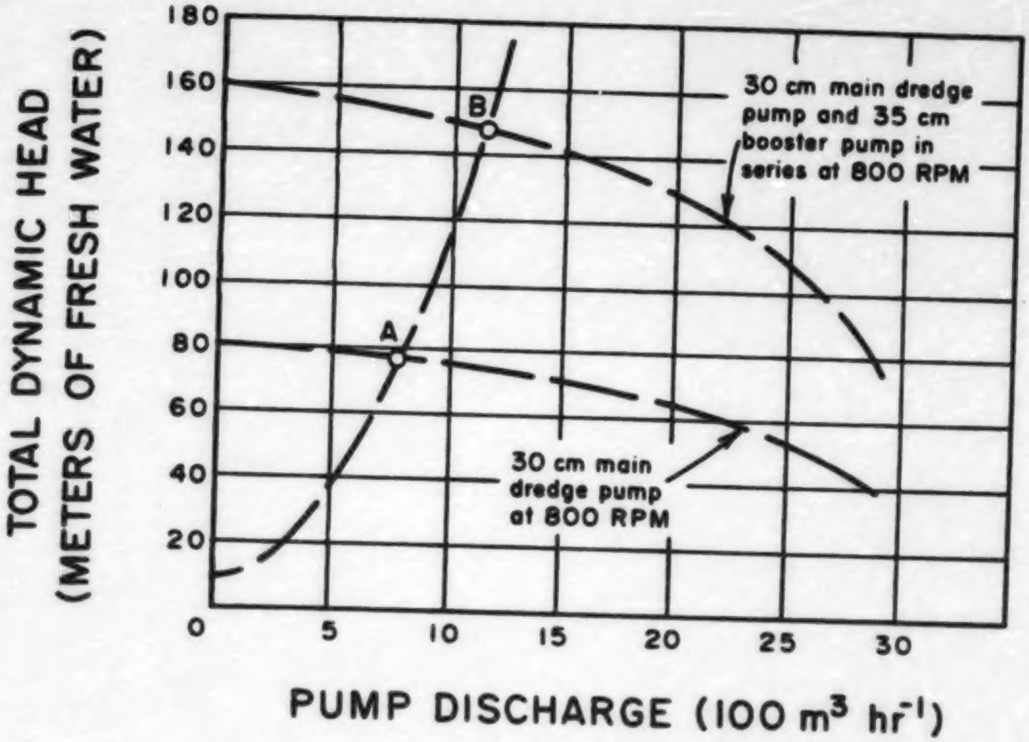


Figure 8.13. Head discharge relationships for a 30 cm dredge pump and a 30 cm booster pump (modified from Pierce, 1970).

$$h_{sv} = 0.8 \text{ m}$$

$$h_{sf} = 0.015 \left[1 + \frac{(20 - 10)}{100}\right] \frac{25.5(3.6)^2}{2(9.82)(0.35)}$$

$$h_{sf} = 0.8 \text{ m}$$

$$H_s = 1.7 + 0.8 + 0.8$$

$$H_s = 3.3 \text{ m}$$

$$h_{de} = 1.2\ (251 - 245)$$

$$h_{de} = 7.2 \text{ m}$$

$$h_{dv} = 1.20\ \frac{(4.9)^2 - (3.6)^2}{2(9.82)}$$

$$h_{dv} = 0.7 \text{ m}$$

$$h_{df} = 0.015\ [1 + \frac{20 - 10}{100} \quad \frac{2416(4.9)^2}{2(9.82)(0.30)}$$

$$h_{df} = 162 \text{ m}$$

$$H_d = 7.2 + 0.7 + 162$$

$$H_d = 169.9 \text{ m}$$

$$\begin{aligned} H_{TDH} &= H_s + H_d \\ &= 3.3 + 169.9 \end{aligned}$$

$$H_{TDH} = 173.2 \text{ m}$$

$$Q = 2826(0.30)^2(4.9)$$

$$Q = 1246 \text{ m}^3 \text{ hr}^{-1}$$

The above calculated head discharge relationship determines one point on the pump system head curve (Figure 8.13). Other points on the curve can be calculated to develop the system curve for plotting. Point 'A' on Figure 8.13 shows the dredge pump alone would produce about 815 m^3 hr^{-1} at a head of 75 m. By rearrangement of equation 8.12 to:

$$V_d = \frac{Q}{\frac{\pi}{4} \times 3600 \; D^2}$$

it can be determined that the discharge pipeline velocity under these conditions would be reduced to slightly more than 3 m sec^{-1}, which is at the lower edge of the efficient operating range (3.0 to 4.0 m sec^{-1}). Point 'B' on Figure 8.13 shows that the dredge pump when operated in series with a second, identical booster pump, would increase the discharge to about 1180 m^3 hr^{-1} at a head of 145 m. Under these conditions the velocity in the discharge pipeline would be increased to a more acceptable 4.6 m sec^{-1}.

Use of a 30 cm booster pump would be quite acceptable. However, if the 30 cm pump is unavailable, what does one do? Let us assume that the only pump available is a 35 cm, high head model. The head discharge curve for this booster pump, together with the 30 cm dredge pump curve, is shown in Figure 8.14. Figure 8.14 also shows the 30 cm/35 cm series operation pump curve and the system head curve for the 1900 m discharge pipeline. Point 'A' on Figure 8.14 represents the capacity of the two pumps in

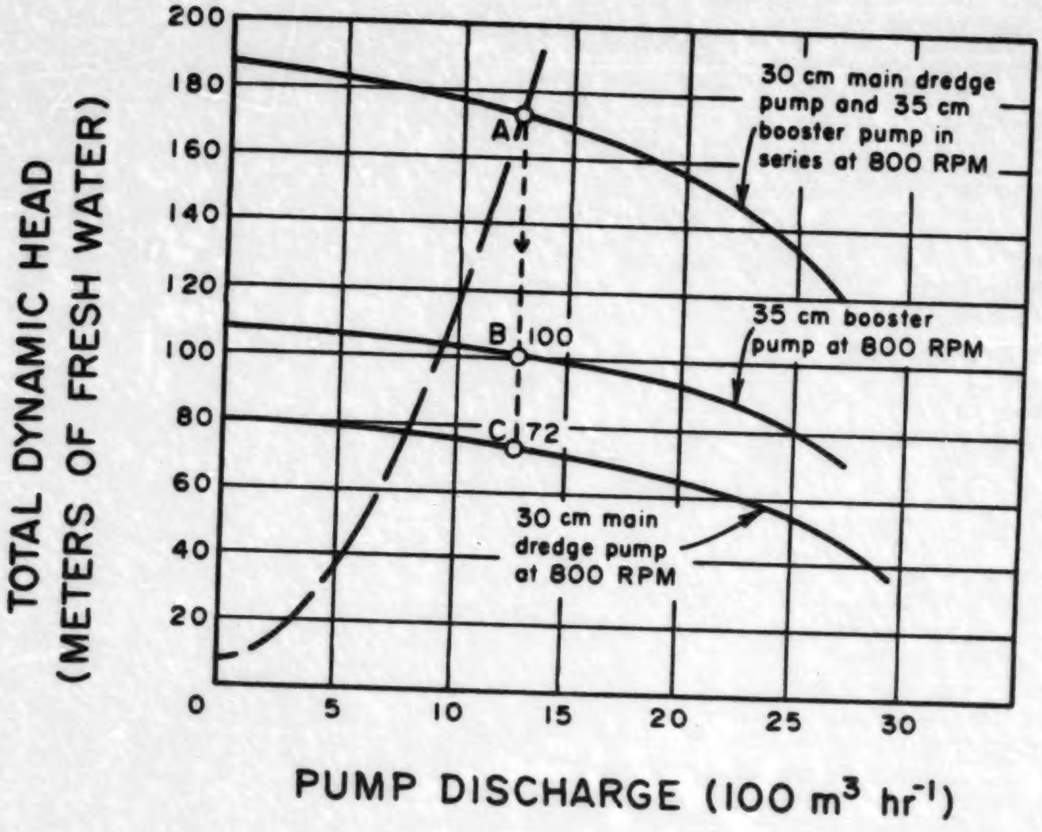


Figure 8.14. Head discharge relationships for a 30 cm dredge pump and a 35 cm booster pump (modified from Pierce, 1970).

series when delivering through the 30 cm diameter, 1900 m long discharge pipeline. The discharge at point 'A' is about 1281 m^3 hr^{-1}, at a head of 172 m. The discharge pipeline velocity at this discharge rate will be about 4.9 m sec^{-1}.

Figure 8.14 can be used to determine what portion of the total head is contributed by each pump. Simply construct a vertical line from Point 'A' downward. The total head for the main pump is 72 m (Point 'C') and that for the booster pump is 100 m (Point 'B'). Added together, the two heads equal the total head of 172 m at operating point 'A'. The continuous brake horsepower needed to operate the two pumps will be:

Dredge Pump

$$BHP = \frac{(1281)(72.0)(1.20)}{(2.737)(55.0)}$$

$$BHP = 735 \times 1.10 = 809 \text{ hp}$$

Booster Pump

$$BHP = \frac{(1281)(100.0)(1.20)}{(2.737)(55.0)}$$

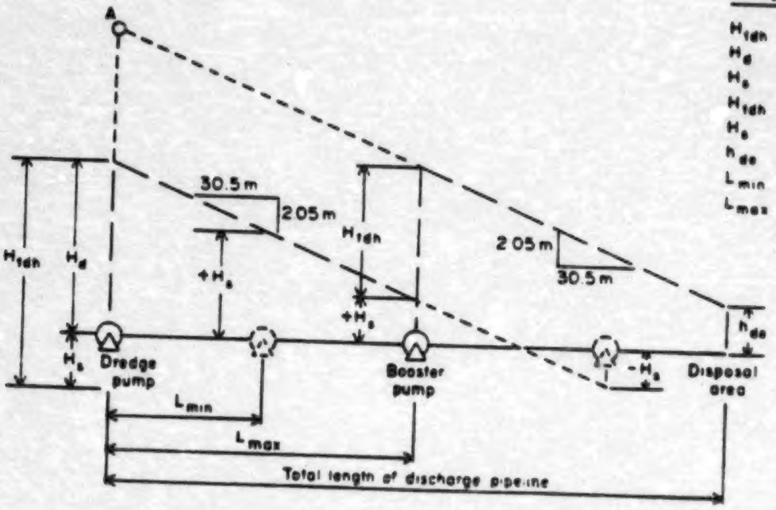
$$BHP = 1021 \times 1.10 = 1123 \text{ hp}$$

The horsepower requirement for the dredge pump is slightly more than the 790 hp that was computed previously. However, it is not enough to pose any problem for operating if one considers the 10 percent factor that was used in the initial selection of the power plant. The booster pump power plant should be selected accordingly in conjunction with a speed reduction gear so that the booster pump runs at 800 rpm.

An energy diagram (Figure 8.15) depicting the heads developed throughout the length of the pipeline will be useful in determining the maximum and minimum allowable distances between the dredge pump and the booster pump. Since there are several variables in the dredgeing process, a positive suction head (H_s) of 10.6 m at the booster pump will be assumed. Also, since there is a positive suction head (H_s) of about 3.3 m on the main dredge pump, that amount must be subtracted from the total dynamic head (H_{TDH}) on the dredge pump to obtain the dredge pump head (H_d), (72.0 - 3.3 = 68.7). From this and the discharge friction head loss per 30.5 m, for 30 cm discharge pipe at a velocity of 4.9 m sec^{-1} (from Figure 8.11), we can compute the maximum pump spacing in equivalent length of pipe as follows:

$$\frac{(68.7 - 10.6)\ 30.5}{2.05} = 864 \text{ m (equivalent length)}$$

The equivalent pipeline length can be converted to actual pipeline length by dividing by the floating pipeline equivalent length correction factor (1.5), which gives an actual pipeline length of 576 m. Since the floating pipeline length is about 760 m, this means the booster pump will have to be barge-mounted on the lake a maximum distance from the dredge of no more than 576 m. Plus and minus signs used with H_s in Figure 8.15 indicate the presence of a suction head or suction lift, respectively. Therefore, it can be seen that a booster pump located at a distance


A
30.5 m
2.05 m
H_{tdh}
H_d
$+H_s$
H_{tdh}
2.05 m
30.5 m
$+H_s$
H_s
Dredge pump
Booster pump
$-H_s$
h_{de}
Disposal area
L_{min}
L_{max}
Total length of discharge pipeline

Energy Diagram Information (m)

H_{tdh}	dredge pump	=	72.0
H_d	dredge pump	=	68.7
H_s	dredge pump	=	3.3
H_{tdh}	booster pump	=	100.0
H_s	booster pump	=	10.6
h_{de}		=	7.2
L_{min}		=	284.0
L_{max}		=	576.0

Figure 8.15. Energy diagram for a 30 cm dredge pump operating in series with a 35 cm booster pump (modified from Pierce, 1970).

greater than L_{max} from the dredge pump will operate under a suction lift. This condition should be avoided.

The minimum spacing between the dredge pump and the booster pump can be computed if the discharge pipeline working pressure is known. We will assume a working pressure of 1.40×10^5 kg m^{-2} which is equivalent to 140 m of waters. The two pumps placed immediately adjacent to one another would produce a discharge head of 168.7 m [(dredge pump head, H_d - dredge pump suction head, H_s) + booster pump total dynamic head, H_{TDH}] is indicated by point 'A' on Figure 8.15. By proportion, the 168.7 m head would result in a discharge pipeline pressure of 1.68×10^5 kg m^{-2}, which exceeds the required working pressure of 1.40×10^5 kg m^{-2}. Therefore, it will be necessary to locate the booster pump some minimum distance from the dredge so that the pipeline friction will reduce the pipeline pressure to a value below the working pressure. This distance should be calculated such that the discharge pressure of the dredge pump plus the booster pump, minus the pipeline friction head loss between the two pumps, is less than the pipeline working pressure. The slope of the energy gradient in Figure 8.15 is nearly constant throughout the length of the discharge pipe. The friction head loss created by the minimum distance between pumps can be computed from formula 8.14.

$$H_l = H_{tdh} + H_d - Wp \qquad (8.14)$$

where:

H_l = head loss in the discharge pipe between the dredge pump and the booster pump (meters of water)

H_{tdh} = booster pump total dynamic head (m)

H_d = dredge pump total head minus the positive suction head (m)

W_p = discharge pipe working pressure (meters of water)

Therefore:

$H_l = 100 + 68.7 - 140$

H_l = 28.7 meters of water

Using the above information, the length of pipeline necessary to create this friction head loss can be computed. Recall from Figure 8.11 that 30 cm discharge pipeline head loss is 2.05 m per 30.5 m of pipeline length at a discharge velocity of 4.9 m sec^{-1}. Therefore:

$$\frac{(30.5)\ 28.7}{2.05} = 427 \text{ m equivalent pipeline length}$$

The equivalent pipeline length can be converted to actual pipeline length by dividing by the equivalent pipeline length correction factor (1.5). Therefore:

$$\frac{427}{1.5} \cong 284 \text{ m}$$

Pierce (1970) used the above type of example to develop two formulas for calculating the maximum and minimum distances required between the dredge pump and the booster pump in order to operate within the prescribed discharge pipeline pressure limits. The formulas should be quite useful in determining these distances when working with smaller size dredges commonly employed in lake restoration. Pierce pointed out that the same approach as described above could be used to determine spacing between pumps if more than one booster pump was required. The two formulas for calculating maximum and minimum spacing requirements, as modified from Pierce (1970) are:

$$L_{max} = \frac{30.5\ (H_d - H_s)}{1.5\ h_{df}} \qquad (8.15)$$

and:

$$L_{min} = \frac{30.5\ (H_d + H_{tdh} - W_p)}{1.5\ h_{df}} \qquad (8.15)$$

where:

L_{max} = maximum distance between the main dredge pump and the booster pump (meters of discharge pipeline)

L_{min} = minimum distance between the main dredge pump and the booster pump (meters of discharge pipeline)

H_d = dredge pump discharge head (meters of fresh water, which equals the discharge pressure gage reading at the dredge pump)

H_s = booster pump suction head (m)

h_{df} = friction loss in the discharge pipeline (meters per 30.5 meters of discharge pipeline, see Figure 8.11

H_{tdh} = booster pump total dynamic head (meters of water, which equals the discharge pressure gage reading at the dredge pump)

Wp = discharge pipe working pressure (meters of water)

From the above analysis for pumping from lake area three to disposal site three, the following can be concluded:

1. The disposal area is beyond the efficient pumping capacity of the dredge pump and a booster pump will be required.
2. Discharge pipeline velocities could be increased to an acceptable level by employing either an identical 30 m booster pump or the available 35 cm model.
3. Head discharge characteristic graphs are highly desirable to assist in booster pump selection since the required head discharge characteristics of the booster are highly dependent on those of the dredge pump.
4. The booster pump should not be located too close to or too far away from the dredge pump. Excessive pipeline pressures will result if it is too close. If it is too far, it may operate under a suction lift, perhaps resulting in pump caviatation, reduced output, and excessive equipment wear.

The above information on hydraulic dredge selection should be helpful to the dredge plan designer in selecting the proper equipment and to the lake manager to assure that the designer has selected the proper equipment. How dredging is actually conducted on a project will depend on the type of equipment selected and on site specific constraints, all of which must be considered when developing the dredge operating plan. The example above can be used as a general guide. However, as stated by Pierce (1970) there is no substitute for actual dredge discharge relationship curves for dredge pumps actually being considered for selection.

It is from these relationships or from pump manufacturers' specifications that the pump suction heads can be obtained.

Disposal Area Design

Once dredge equipment is selected, the critical concern of the dredge plan designer is the disposal area. Upland disposal has become a common practice. The challenge is to design and construct a containment/disposal area of adequate size and retention time to hold the dredged material volume and to reduce suspended solids concentrations to meet effluent requirements. According to Montgomery (1982),the chief deterrents to the successul operation of disposal areas is nonuniform lateral distribution of flows and short-circuiting currents within the disposal basin. This is due primarily to the fact that relatively little is known about the actual sedimentation process in dredged material containment areas. The hydrodynamic properties of particles in dredged materials do not behave according to Stoke's Law for a single particle falling unobstructed through liquid. Many of the lake sediment particles are of near colloidal size; once resuspended they do not resettle rapidly.

Field investigation of the dredge site must be conducted to obtain disposal area design information. The most basic piece of field information is an estimate of the inplace sediment volume as described previously. Two types of laboratory tests will also need to be conducted in order to develop information for the disposal area design. The first of these are sediment characterization tests including natural water content, Atterberg limits, organic content, and specific gravity for fine grain sediments. Grain size analyses are adequate for course grained sediment. The second type of test is to determine sedimentation rate. Montgomery (1978) demonstrated that freshwater dredge slurries containing solids concentrations up to 175 g l^{-1} could be characterized by flocculent settling tests, where particles agglomerate during settling, thus producing changes in physical properties and settling rates. Therefore, many hydraulically dredged lake sediments could be characterized by this test procedure.

Montgomery (1978) prefaces the flocculent settling tests procedures with the statement that, "if an interface forms near the top of the settling column during the first day of the test, sedimentation is governed by zone settling, and that test procedure should be initiated." He indicated that zone settling might prevail at solids concentrations in excess of 175 g l^{-1}. Therefore, the zone testing procedure in which the flocculent suspension forms a lattice structure and settles as a mass, is described also. The two procedures are the following.

Flocculent Settling Procedure

1. A settling column such as that shown in Figure 8.16 is used. The test column depth should approximate the effective settling depth of the proposed containment area. A practical test depth is 2 m. The column should be at least 20 cm in diameter with sample ports at 0.3 m intervals. The column

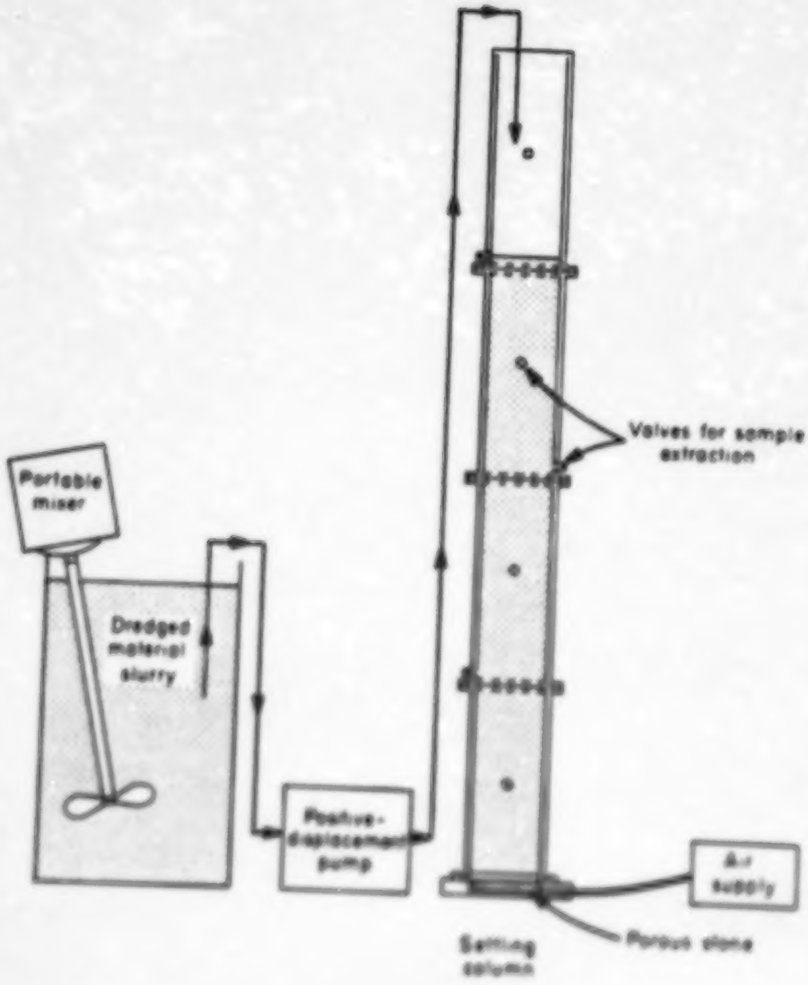

Portable mixer
Dredged material slurry
Positive-displacement pump
Valves for sample extraction
Air supply
Porous stone
Settling column

Figure 8.16. Schematic of fine grained sediment settling test equipment (modified from Montgomery, 1978).

should have provisions to bubble air from the bottom to keep the slurry mixed during the column-filling period.

2. Mix the sediment slurry to the desired suspended solids concentration in a container with sufficient volume to fill the test column.
3. Pump or pour the slurry into the test column using air to maintain a uniform concentration during the filling period.
4. While the column is completely mixed, draw off samples at each sample port and determine the suspended solids concentration. Average these values and use the results as the initial concentration at the start of the test. After the initial samples are taken, stop the air bubbling and begin the test.
5. Allow the slurry to settle, then withdraw samples from each sampling port at regular time intervals and determine the suspended solids concentrations. Sampling intervals depend on the settling rate of the solids -- usually at 30-minute intervals for the first 3 hours and then at 4-hour intervals until the end of the test. Continue the test until the interface of solids can be seen near the bottom of the column and the suspended solids level in the fluid above the interface is < 1 g l^{-1}.
6. If an interface has not formed within the first day on any previous tests, run one additional test with a suspended solids concentration sufficiently high to induce zone settling behavior. This test should be carried out according to the procedures outlined below for zone settling tests.

The exact concentration at which zone settling behavior occurs depends upon the sediments being used to estimate the volume required for dredged material storage.

Zone Settling Test Procedure

This test procedure consists of placing a slurry in a column similar to Figure 8.16 and recording the fall of the liquid-solids interface over time. The depth to the interface is then plotted as a function of time. From this plot, the slope of the constant settling zone of the curve represents the zone settling velocity, which is a function of the initial test slurry concentration. Information needed to design a containment area when zone settling characteristics prevail can be obtained by using the following procedure from Montgomery (1978):

1. Use a settling column such as that shown in Figure 8.16. It is important that the column diameter be sufficient to reduce wall effects and the test be performed at a slurry depth near that expected in the field. Therefore, a one-liter graduated cylinder should never be used to perform a zone settling test for sediment slurries representing dredging-disposal activities.
2. Mix the slurry to the desired concentration and pump or pour it into the test column. Test concentrations should range from about 60 to 200 g l^{-1}. Air may not be necessary to keep the slurry mixed if the filling time is less than 1 minute.

3. Record the depth to the solid-liquid interface with respect to time. Observations must be made at regular intervals to gain data for plotting the depth to interface versus time curve. It is important to make enough observations to clearly define this curve for each test.
4. Continue the readings until sufficient data are available to define the maximum point of curvature of the depth to interface versus time curve for each test. The tests may require from 1 to 5 days to complete.
5. Perform a minimum of eight tests. Data from these tests are required to develop the curve of zone settling velocity versus concentration.
6. The test should be continued for a period of at least 15 days to provide data for estimating volume requirements.

Design Procedures

Montgomery (1978) has described a complete procedure for designing dredged material disposal areas based on field and laboratory observations. He describes the methods for both saltwater sediments and freshwater sediments, which are based on zone settling properties and flocculent settling properties, respectively. Since this text is concerned with freshwater lake sediments, and most of them are expected to be governed by flocculent settling properties, only that portion of Montgomery's design procedure will be described.

Estimate *in situ* Sediment Volume

The initial step in any proposed dredging project is to estimate the amount of sediment to be removed. Methods for doing this were described earlier in this chapter. Any procedure which provides an accurate estimate for computing disposal volume and cost is satisfactory.

Determine Physical Characteristics of the Sediments

The information required includes the natural water content of the sediment (*in situ* water content), Atterberg limits, organic content, specific gravity, and grain size analyses. Montgomery (1978) recommended the methods described by Palermo *et al*. (1978) for these determinations. Fine grained sediment samples obtained with a Petersen Dredge are considered to be adequate for defining the *in situ* water content of sediment. This determination is critical to the disposal area design because sediment water content of representative samples, w, is used to determine the *in situ* void ratio, e, from the following equation:

$$e_i = \frac{(w/100)\ G_s}{S_d/100} \qquad (8.17)$$

where:

w = water content of sediment (%)

Gs = specific gravity of sediment solids

S_d = the degree of saturation (equal to 100% for sediment)

Analyze Proposed Dredging and Disposal Data and Laboratory Sedimentation Tests

Once tentative dredge selection has been made, this information can be used in the disposal area design. For example, the designer must estimate the containment area influent rate, influent suspended solids concentration, effluent rate (for weir sizing), effluent suspended solids concetration allowed, and time required to complete the disposal activity. If information, as previously described in this chapter, is unavailable, one must assume the largest size dredge that could be used on the project to assure the disposal area is not undersized. Montgomery (1980) has recommended the use of a suspended solids concentration of 145 $g\ l^{-1}$ (13% by weight) to be used for design purposes if the actual concentrations are unknown. That figure, however, is derived from experience in maintenance dredging of waterways (freshwater), not from dredging lake type sediments. Therefore, the figure should be used with caution when dealing with flocculent lake sediments. Sedimentation tests for freshwater should be performed at this point according to the method described previously.

Design Method for Freshwater Sediments

According to Montgomery (1978) the following design method should be sufficient to remove suspended solids from freshwater disposal area discharges down to a level of 2 $g\ l^{-1}$. If that level is insuficient to meet discharge requirements it will be necessary to treat the discharge.

Because larger particle flocs settle at faster rates overtaking finer flocs in their descent, the increased contact increases the floc size and enhances settling rates. Therefore, the greater the ponding depth in the disposal area, the greater the probability for contact among sediments and flocs and the more efficient the settling with increased depths.

Montgomery (1978) conducted laboratory settling rates on freshwater sediments and arranged the data as shown in Table 8.4. From this point he prescribed the following procedure:

1. Arrange data from laboratory tests illustrated in Table 8.4 into the form in Table 8.5.
2. Plot these data as shown in Figure 8.17. The percent by weight of initial concentration for each depth and time is given in Table 8.5. The solid curved lines represent the concentration depth profile at various times during settling (refer to Figure 8.17). Numbers appearing along the horizontal depth lines are used to indicate area boundaries.
3. Compute a design concentration using data from the 15-day zone settling test.

The design concentration, C_d, is defined as the average concentration of the dredged material in the containment area at the end of the disposal activity and is estimated from results of the 15-day column settling test. The following steps can be used to compute the C_d:

Table 8.4. Observed flocculent settling concentrations with depth, g/l (modified from Montgomery, 1978).*

Time (min)	Depth From Top of Settling Column (m) 0.3	0.6	0.9	1.2	1.5	1.8
0	132	132	132	132	132	132
30	46	99	115	125	128	135
60	25	49	72	96	115	128
120	14	20	22	55	78	122
180	11	14	16	29	75	119
240	6.8	10.2	12	18	64	117
360	3.6	5.8	7.5	10	37	115
600	2.8	2.9	3.9	4.4	14	114
720	1.01	1.6	1.9	3.1	4.5	110
1020	0.90	1.4	1.7	2.4	3.2	106
1260	0.83	1.14	1.2	1.4	1.7	105
1500	0.74	0.96	0.99	1.1	1.2	92
1740	0.63	0.73	0.81	0.85	0.94	90

* Data from actual test on freshwater sediments.

Table 8.5. Percent of initial concentration with time (modified from Montgomery, 1978).*

Time (min)	Depth From Top of Settling Column (m) 0.3	0.6	0.9
0	100	100	100
30	35	75	87
60	19	37	55
120	11	15	17
180	8	11	12
240	5	8	9
360	3	4	6
600	2.0	2.2	3.0
720	1.0	1.2	1.4

* Initial suspended solids concentration = 132 g/l.

1. Compute concentration versus time for the 15-day settling test. Assume zero solids in the water above the solids interface to simplify calculation.
2. Plot concentrations versus time on log-log paper.

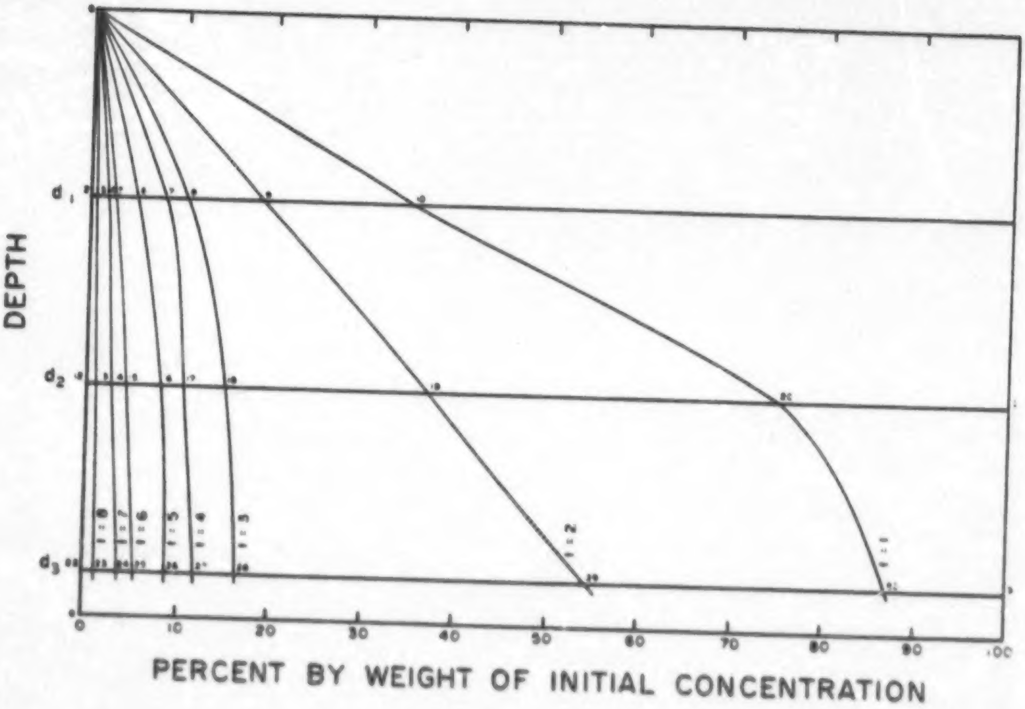


Figure 8.17. Percent of initial concentration versus depth profile for flocculating dredged material particles (modified from Montgomery, 1978).

3. Draw a straight line through the data points. This line should be drawn through the points representing the consolidation zone.
4. Estimate the time of dredging by dividing the dredge production rate into volume of sediment to be dredged.
5. Estimate the concentration of time $t_{\frac{1}{2}}$ (one-half the time required for the disposal activity determined in Step 4) using the figure developed in Steps 2 and 3. This time is an approximation of the average time of residence for the dredged material in the containment area. Since concentration is a function of time, one-half the dredging time would represent a period during which one-half of the dredged material would have been in the area longer and the other half less than a time equal to one-half the dredging time.
6. Use the value computed in Step 5 as the design solids concentration, C_d.

Computation of Detection Time Required for Sedimentation

1. Calculate removal percentage at depths of 0.30, 0.60, and 0.90 m for various times using the plot illustrated in Figure 8.17. The removal percentage for depth d_1 and t = 1 is computed from formula 8.18.

$$R = \frac{\text{Area } 0,\ 10,\ 11,\ 1^*}{\text{Area } 0,\ 2,\ 11,\ 1} \times 100 \qquad (8.18)$$

where:

R = the removal percentage

Area numbers = the numbers corresponding to the numbers used in Figure 8.17 to indicate the area boundaries for the total area down to depth, d (0, 2, 11, 1) and the area to the right of the, t = 1 line (0, 10, 11, 1).

2. Plot the solids removal percentages versus time as shown in Figure 8.18.
3. Theoretical detention times can be selected from Figure 8.18 for various solids removal percentages. Select the detection time, T, that gives the desired removal percentage for the design ponding depth.
4. Montgomery determined experimentally that a correction factor ranging from 2.13 to 2.72 needs to be applied to the theoretical detention time, T, because of short-circuiting and dispersion of flows through the disposal basin. Therefore, he recommended application of a median correction factor of 2.25 as follows:

$$T_d = 2.25\ T \qquad (8.19)$$

where:

T_d = the design detention time

T = the theoretical detention time

The above procedures are aimed at providing sedimentation basins with sufficient areas and detention times to accommodate continuous, hydraulic dredge disposal activities while also

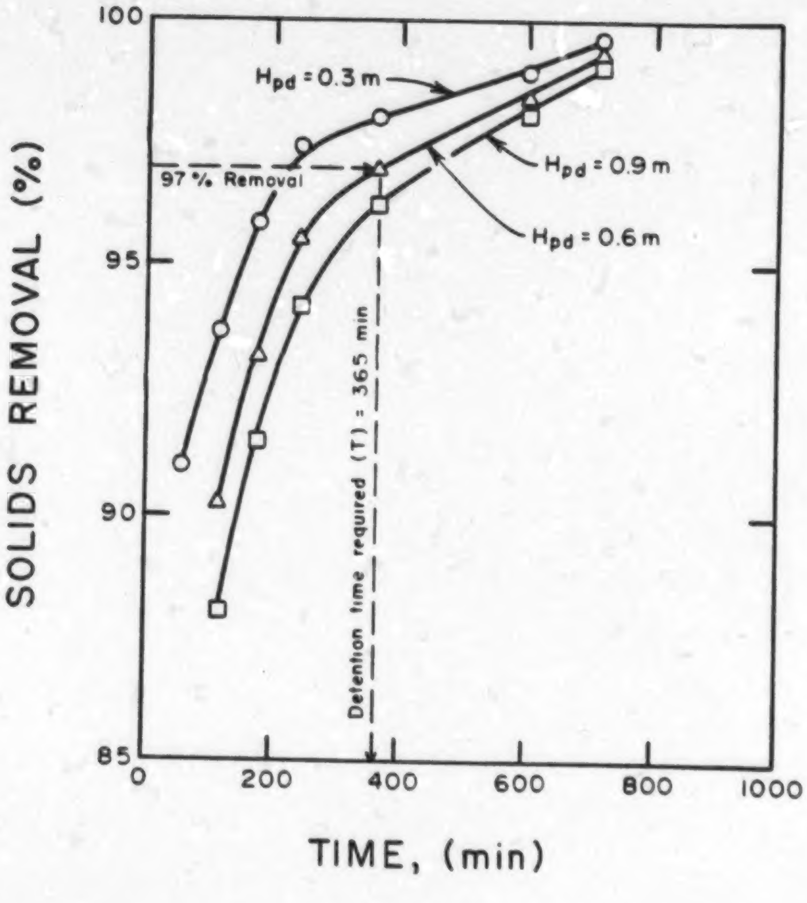


Figure 8.18. Solids removal versus time as a function of depth (modified from Montgomery, 1978).

meeting effluent suspended solids requirements. In addition, the basins must be designed to meet volume requirements of the job. The sedimentation basin total volume requirement includes volume for storage of dredged material, volume for sedimentation (ponding depths), and free-board volume (volume above water surface). Coarse-grained material storage volume (> 200 sieve) must be determined separately since this material behaves independently of the fine grained (< 200 sieve) material.

Estimate Volume Occupied by Dredged Material in the Sedimentation Basin

The dredged material volume in the sedimentation basin following completion of disposal activity can be computed as follows.

1. Compute the average void ratio of fine-grained dredged material in the sedimentation basin at the completion of the dredging operation using the design concentration determined in earlier steps as dry density of solids. Use the following equations to determine void ratio:

$$e_o = \frac{G_s \gamma_w}{\gamma_d} - 1 \qquad (8.20)$$

where:

e_o = average void ratio of dredged material in the sedimentation basin at the completion of the dredging operation

G_s = specific gravity of sediment solids

γ_w = density of water (g l^{-1})

γ_d = dry density of solids at design concentration (C_d = γ_d), (g l^{-1})

2. Compute the changes in volume of fine-grained sediments after disposal in the sedimentation basin from:

$$\Delta V = V_i \frac{e_o - e_i}{1 + e_i} \qquad (8.21)$$

where:

ΔV = change in volume of fine-grained sediments after disposal in the sedimentation basin (m^3)

e_i = average void ratio of *in situ* sediments

V_i = volume of fine-grained sediments (m^3)

3. Compute the volume required by dredged material in the sedimentation basin from:

$$V = V_i + \Delta V + V_{sd} \qquad (8.22)$$

where:

V = volume of dredged material in the sedimentation basin at the end of the dredging operation (m^3)

V_{sd} = volume of sand (compute using 1:1 ratio), (m^3)

Estimate Basin Depth

The above procedures have provided a design area, A_d, and design detention time, T_d, required for fine-grained dredged material sedimentation. Equations 8.20 - 8.22 are used to estimate the volume and corresponding depth requirements for the storage of solids in the containment area. The existing topography and surficial geology of the proposed containment must be kept in mind throughout the design process since they can have a significant affect on the average depth of the containment area. The following procedures were recommended by Montgomery (1978) to estimate the thickness of dredged material at the end of disposal when freshwater sediments are being disposed.

1. Compute the volume required for sedimentation from:

$$V_B = Q_i T_d \tag{8.23}$$

where:

V_B = sedimentation basin volume required to meet suspended solids effluent requirements (m^3)

Q_i = influent rate (m^3 hr^{-1}), $Q_i = A_p V_d$; assume $V_d = 4.6$ m sec^{-1} in absence of data and convert Q_i calculated in m^3 sec^{-1} to m^{-3} hr^1; A_p is the cross-sectional area of the dredge discharge pipe (m^2) and V_d is the velocity of dredge discharge (m sec^{-1})

T_d = required design detention time from equation 8.19

2. Consult with soils design engineers to determine maximum height allowable for confining dikes, D. It might be desirable to use less than maximum allowed dike heights in soft soils.

3. Compute the required design area as a minimum required surface area for solids storage from:

$$A_d = \frac{V}{H_{dm(max)}} \tag{8.24}$$

where:

A_d = the design basin surface area (m^2)

V = volume of dredged material in the sedimentation basin at the end of the dredging operation (m^3), from equation 8.22

$H_{dm(max)} = D - H_{pd} - H_{fb}$ (m); when D is the dike height (m), H_{pd} is the average ponding depth over the area (m), with a minimum of 0.6 m recommended, and H_{fb} is the freeboard above the basin water surface to prevent wave overtopping and subsequent damage to the confining earth dikes (m), with a minimum of 0.6 m recommended to account for fetch and wind, or else wet the design area, A_d equal to the known surface area for existing sites.

4. Evaluate volume available for sedimentation near the end of the disposal operation from equation 8.25.

$$V^* = H_{pd}A_d \tag{8.25}$$

where:

V^* = volume available for sedimentation near the end of the disposal operation (m^3)

H_{pd} = average ponding depth over the area (m)

A_d = the design basin surface area (m^2)

5. Compute V^* and V_B. If the volume required for sedimentation is larger than V^*, the sedimentation basin will not meet the suspended solids effluent requirements throughout the disposal operation. If that is the case, one of the following measures can be taken to ensure that effluent suspended solids concentrations are met:
   a. increase the design area, A_d;
   b. operate the dredge intermittently when V^* becomes less than V_B or use a smaller size dredge;
   c. provide for post-treatment of effluent to remove solids (see Barnard and Hand, 1978).
6. Estimate the thickness of dredged material at the end of the disposal operation using equation 8.26.

$$H_{dm} = \frac{V}{A_d} \tag{8.26}$$

where:

H_{dm} = thickness of the dredged material layer at the end of the disposal operation (m)

V = volume of the dredged material in the basin (m^3), from equation 8.22

A_d = design area (m^2), from the equation $A_d = 2.25A$, When A_d is the design basin surface area (m^2), and A is the containment area surface requirement (m^2), as computed from equation 8.27:

$$A = \frac{Q_i C_i}{S_L} \tag{8.27}$$

where:

A = containment area surface requirement (m^2)

Q_i = influent rate (m^3 hr^{-1}), as shown for equation 8.23

C_i = influent solids concentration (g l^{-1}), or use 145 g l^{-1} if no data are available, but note the previous caution for this concentration

S_L = design solids loading (kg hr^{-1} m^2)

As noted in step 4, of Computation of Detention Time Required for Sedimentation, a factor of 2.25 must be applied to the time to account for short-circuiting and dispersion of flows through the disposal basin. Since detention time is in direct relationships to the surface area of the disposal basin, it follows that the above computed containment area surface requirement also must be increased by a factor of 2.25. Therefore:

$$A_d = 2.25A \tag{8.28}$$

where:

A_d = design basin surface area (m^2)

A = area determined from equation 8.27 (m^2)

Sediment exhibiting zone settling characteristics requires a different disposal area design method. If this should be encountered, the reader is referred to Montgomery (1978 and 1982). The above disposal area design guidance is the best available at this time. However, some have expressed the opinion that it may be inadequate to address the extremely flocculent, organic sediments found in lakes. Dunst (1984) reported that the specific gravity of sediment from Lilly Lake, Wisconsin, was on 1.02. Therefore, one should be extremely cautious about relying completely on the above disposal area design procedures.

(7) Case Histories

Dunst et al. (1974) identified 33 completed, or in progress, lake dredging projects in 1974. Seven years later Peterson (1981) listed 64 such projects. Both the 1974 and 1981 listings probably were incomplete since many small lake dredging projects are completed but never reported in the open literature. If records are available from these projects there is little data collected beyond the completion date. Therefore, few well documented long-term evaluations are available. The following case histories represent relatively well documented projects. They are presented for that reason and because each has achieved its objective with somewhat different approaches. Peterson (1981) has described additional case histories.

It should not be construed from the following presentations that all dredging projects to control internal nutrient recycling are successful. On the contrary, there have been failures. However, almost all failures or marginally successful projects can be traced to inaccurate predredging assessment of the problem and to inadequate amounts of sediment removal (Brashier et al., 1973; Churchill et al., 1975; George et al., 1981; Ryding, 1982). The need for accurate implementation assessment of the problem cannot be overemphasized.

Lake Trummen, Sweden

The most thoroughly documented, long-term evaluation of lake dredging took place at Lake Trummen, Sweden. The lake, located near the city of Växjö, began receiving domestic waste discharge from the kitchen of St. Sigfrid's Hospital in 1895 (Sjön Trummen i Växjö, 1977). Modern toilets served by septic tanks near the lakeshore were installed in 1936. In 1943, a flax mill began discharging to the 400 ha lake. From that time winter fish kills became common and the quality of the lake declined rapidly. Sewage discharge to the lake stopped in 1959 after the flax-dressing plant closed and wastewater from the surrounding area was connected with a municipal treatment system. Despite the wastewater diversion, no apparent recovery of the lake followed. In fact, its condition became so bad in the early 1960s that citizens of Växjö considered filling the basin (Bjork, 1974). Limnological investigations conducted by the University of Lund from 1966 to 1969 revealed that the upper one meter of sediment was extremely rich in nutrients relative to that underlying it.

One-half meter of sediment was dredged from the main lake basin in 1970 and another one-half meter was removed in 1971 (Bjork, 1974). This removal increased the mean depth of the lake from 1.1 to 1.75 m and the maximum depth from 2.1 to 2.5 m. The total volume of the sediment removed was about 30 x 10^5 m^3. By any standard, the lake was still extremely shallow and could have remained eutrophic except that the sediment skimming treatment reduced the phosphate phosphorus content in the surface layer of sediment from approximately 2.4 to 0.1 mg $liter^{-1}$ (Sjön Trummen i Växjo, 1977).

The dredged material was partially disposed in shallow diked-off bays. The remainder went to upland diked ponds where return flow was treated with aluminum sulfate to reduce the total phosphorus concentration from about 1 mg $liter^{-1}$ to 30 μg $liter^{-1}$. Dried dredge material was sold as top soil dressing for about $2 (U.S.) m^{-3}.

Bengtsson et al. (1975) indicated that the role of the sediment in recycling nutrients was reduced substantially following dredging. The data in Figure 8.19 demonstrate an approximate 90 percent reduction of total phosphorus in the surface water of Lake Trummen after dredging (from 600 μg $liter^{-1}$ to a range of 70-100 μg $liter^{-1}$). Phosphorus reductions are especially noticeable in summer. Total nitrogen concentrations were also reduced

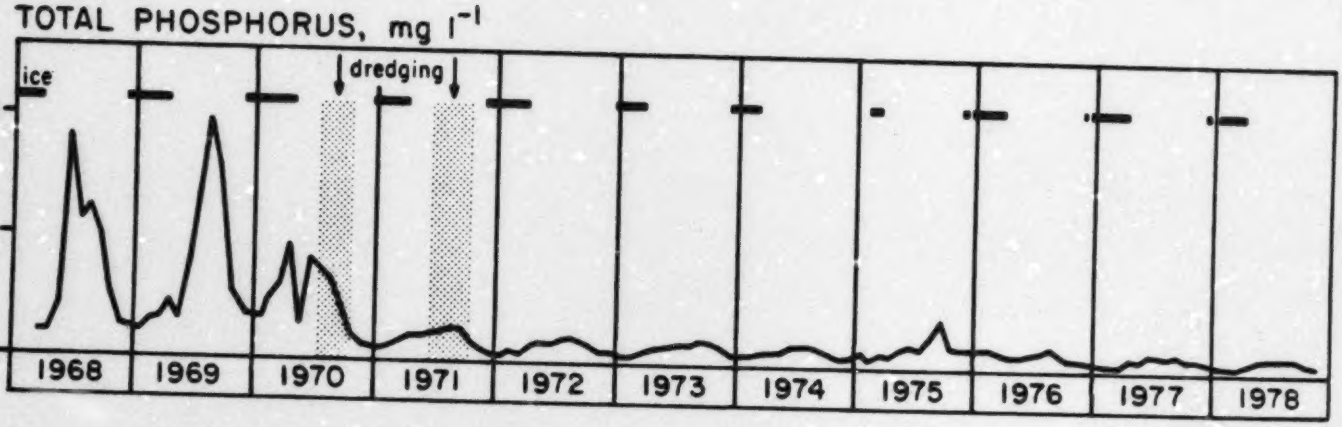


Figure 8.19. Total phosphorus concentration in Lake Trummen, Sweden, before and after dredging (courtesy of Gunnar Andersson, Department of Limnology, University of Lund, Lund, Sweden).

approximately 80 percent from 6.3 mg $liter^{-1}$ to 1.3 mg $liter^{-1}$ (Andersson, 1981). The temporary increase of phosphorus in 1975 (Figure 8.19) was associated with a large influx of planktivorous cyprinid fish (Andersson _et_ _al_., 1978). This type of fish population is generally considered to be undesirable and associated with reduced water quality. During 1976 about 2 metric tons (30 kg ha^{-1}) of fish were taken from the lake. The practice continued through 1979 and phosphorus concentrations remained low. When fishing stopped in 1979 the P levels began to rise again (Andersson, 1981). This suggests that Shapiro _et_ _al_. (1975) may have been correct when they recommended that symptoms of eutrophication in lakes might be controlled by fish populations.

The significant nutrient reduction in the lake resulted in equally significant biological changes. The Shannon diversity index for phytoplankton rose from 1.6 in 1968 to 3.0 in 1973 (Cronberg _et_ _al_., 1975). Secchi disc transparency went from 23 to 75 cm during the same period. The blue-green algal biomass was reduced dramatically and the nuisance species _Oscillatoria_ _agardii_ disappeared completely. Phytoplankton productivity went from a level of 370 g C m^{-2} in 1968-69 to about 225 g C m^{-2} in 1972-73.

The effect of dredging on the benthic community of Lake Trummen was negligible. A year after dredging tubificid oligocheates and chironomids were more numerous than before dredging, but the total number of benthic organisms changed little (Andersson _et_ _al_., 1975). Rapid recolonization was attributed to the mobility and constant swarming of chironomids.

Surface sediment skimming at Lake Trummen was successful in rolling back the effects of eutrophication. The results have remained relatively constant for nine years, which attests to the thorough pretreatment evaluation and the rational plan for restoration. These results are highly encouraging, because if surface sediment dredging is successful in a shallow lake like Trummen, it also should be successful in deeper ones with similarly enriched surface sediment.

Lilly Lake, Wisconsin

Lilly Lake is a 37 ha closed basin lake in southeastern Wisconsin. Its agricultural watershed is only 155 ha. The lake suffered for several years from infilling by floating leafed pond weeds (Potamogeton amplifolius). In 1977, shoaling had advanced to the point that the lake had a maximum depth of 1.8 m and a mean depth of only 1.4 m. The basin contained more than 10 m of partially decomposed plant materials. The Wisconsin Department of Natural Resources (WDNR) (1969) reported that chemical eradication and restocking of centrarchid and northern pike failed due to severe winter kill problems. The rate of basin infilling by sedimentation had reached 0.5 cm yr^{-1} (Dunst, 1981).

Restoration of Lilly Lake for fish management purposes was recommended by the Wisconsin DNR (1969). The plan called for deepening at least 10 percent of the basin to a depth of 6 m. This required removing 665 x 10^3 m^3 of sediment. Use of a hydraulic suction dredge was proposed, with the idea that sediment would flow to the dredge head as a cone of depression was created. However, the lightweight sediment (specific gravity = 1.02) had much greater cohesiveness than the pre-dredging assessment indicated and a cutterhead had to be employed.

Dredging began in July 1978 and continued until near freeze-up at the end of October. It began again in May 1979 and was completed at the end of August. Most of the dredged material was pumped approximately 3 km to an abandoned gravel pit. Although the groundwater table rose temporarily due to this disposal, it resumed normal levels shortly after disposal ceased in both 1978 and 1979 and monitoring wells revealed no adverse impact due to chemicals (Dunst et al., 1984). Figures 8.20, 8.21, 8.22, and 8.23 show in-lake changes for selected variables before, during, and after dredging. Prior to dredging in 1977 the total inorganic nitrogen (TIN) concentrations were near detection limits. Although total phosphorus (TP) increased to more than 40 µg liter^{-1} during July, August, and September before dredging it was not due to excessive phytoplankton concentrations as evidenced in Figure 8.22. The increases may have been due to decomposition of sloughed macrophyte parts and filamentous algae on the bottom. During dredging much of the increase was due to resuspended sediment. Despite previously reported winter kills of fish, the dissolved oxygen (DO) concentrations remained above 6 mg liter^{-1} throughout the study except for February and March of 1979. These DO declines were preceded by large increases in TIN and TP in late fall of 1978. The DO depletion may have resulted from oxygen demand of suspended organics resulting from dredging into the late

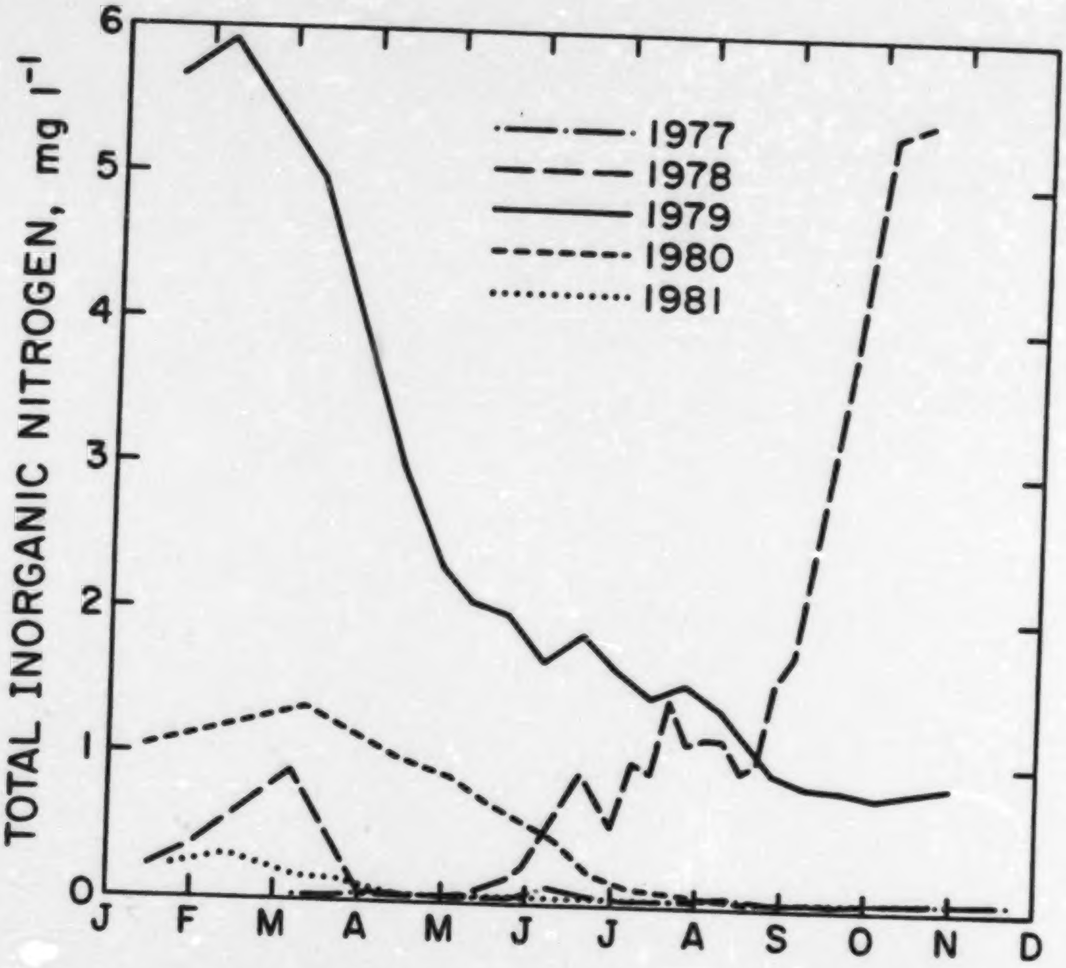


Figure 8.20. Total inorganic nitrogen in Lilly Lake, Wisconsin (data courtesy of Russell Dunst, Wisconsin Department of Natural Resources, Madison, Wisconsin).

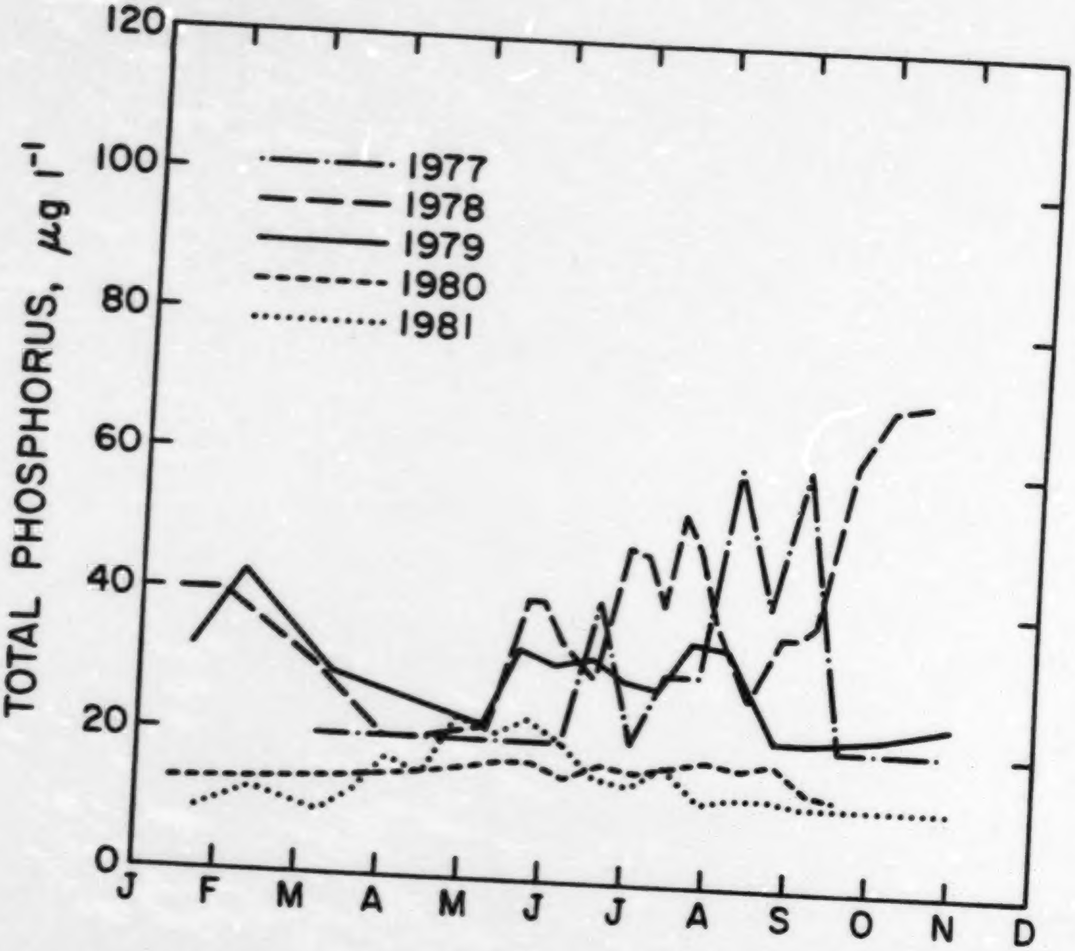


Figure 8.21. Total phosphorus in Lilly Lake, Wisconsin (data courtesy of Russell Dunst, Wisconsin Department of Natural Resources, Madison, Wisconsin).

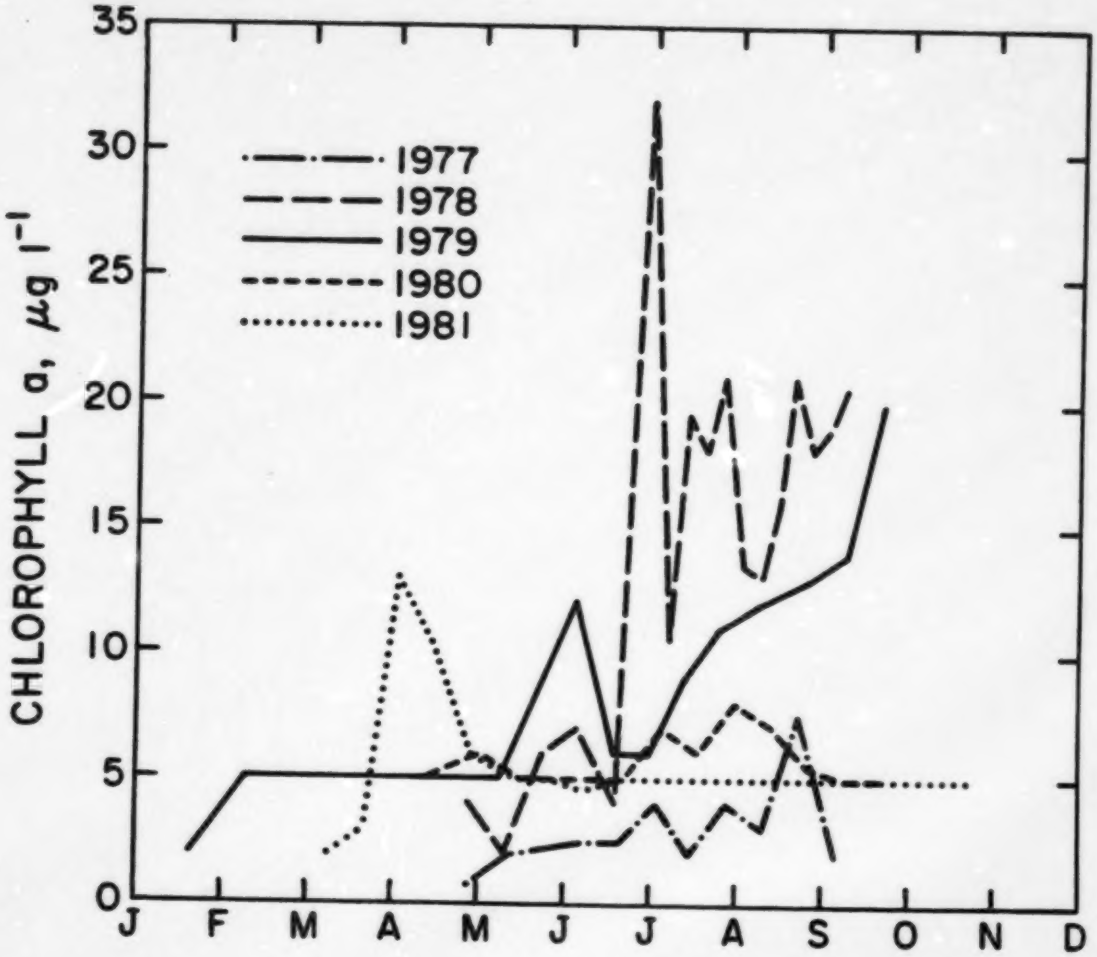


Figure 8.22. Chlorophyll a in Lilly Lake, Wisconsin (data courtesy of Russell Dunst, Wisconsin Department of Natural Resources, Madison, Wisconsin).

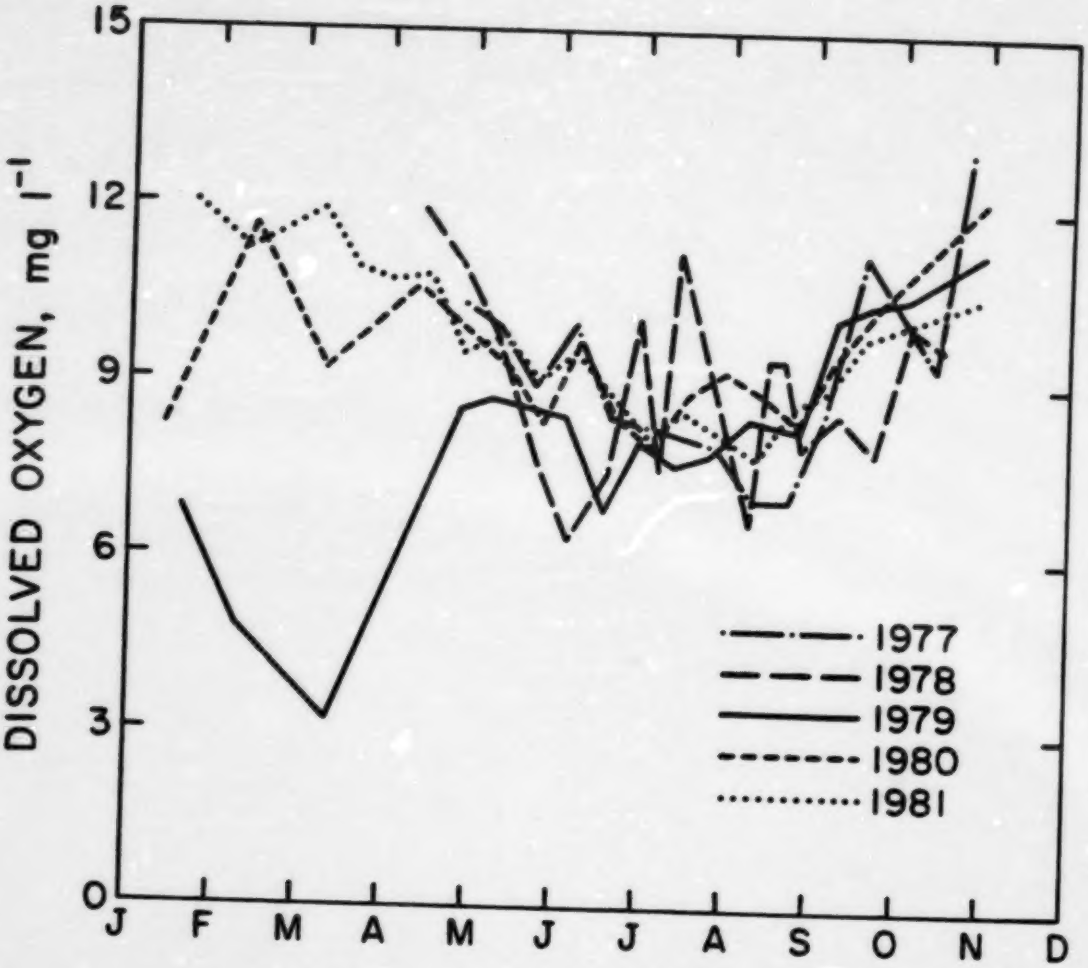


Figure 8.23. Dissolved oxygen in Lilly Lake, Wisconsin (data courtesy of Russell Dunst, Wisconsin Department of Natural Resources, Madison, Wisconsin).

fall. There was no similar oxygen decline the following year when dredging stopped in August. The water quality *per se* of Lilly Lake was not exceptionally poor prior to dredging. However, there was so little water in the lake (1.4 m) due to in-filling by macrophytes and partially decomposed macrophytes that recreational use was severely impaired.

Adverse, although not severe, short-term changes occurred during the active dredging period. TIN increased rapidly (Figure 8.20) when dredging began in July 1978, due mostly to the liberation of ammonium-nitrogen from the sediment (Dunst, 1981). It remained in excess of 3.5 mg liter^{-1} during most of the 1978-1979 ice-cover period then declined steadily throughout the spring and summer even though dredging ended in November 1978 and began again in May 1979.

The TP concentrations behaved in a fashion similar to TIN, except there were minor peaks (lower than predredging, 1977) in TP during the summer of 1978. The TP remained relatively high during the fall and winter of 1978-1979, probably due to increased levels of suspended solids in the water column due to dredging. This is confirmed in part by elevated turbidity and BOD levels during the same period (Dunst *et al.*, 1984).

Phytoplankton responded in a predictable manner to the increased nutrient concentrations (Figure 8.22). The July-September average gross primary productivitity increased from 185 mg C m^{-3} day^{-1} in 1976 and 140 mg C m^{-3} day^{-1} in 1977 to over 1,000 mg C m^{-3} day^{-1} in 1978 (Dunst, 1981).

With the completion of dredging in September 1979, all of the above variables returned to near or below predredging levels. The 1980 and 1981 TP concentrations were significantly lower than they were prior to or during dredging. Chlorophyll a concentrations were slightly higher in 1980 and 1981 than in the predredging year of 1977, however, the increase was relatively insignificant (approximately 3-4 μg liter^{-1}). More importantly, the water storage capacity (basin volume) of the lake was increased 128 percent (Dunst, 1981) and the overall objective of increasing the recreational potential of Lilly Lake appeared to have been realized. The two-summer inconvenience of dredging resulted in what lake users hoped would be a long-term benefit.

However, dredging has not resulted in complete success relative to creating a weed free lake. High water transparency (2.2 m) in 1981 permitted plant growth to a depth of approximately 3.7 m. This is in total agreement with the maximum depth of growth (3.5 m) predicted by equation 8.1 (Dunst *et al*., 1984). Plant growth over 75 percent of the lake basin was dominated by *Chara* (a macrophytic algae) and *Myriophyllum* spp. The plant growth was particularly offensive since it grew to the surface in many areas. In 1982, plant growth covered the same area, but was less offensive since it remained 1.2 to 1.8 m below the water surface except in the nearshore areas. While the 1980 and 1981, macrophyte biomass was considerably less (about 100 g m^{-2}) than during the predredging period (685 and 335 g m^{-2} in 1976 and 1977, respectively), this rapid reinvasion poses a major concern regarding the effectiveness of dredging tc control macrophytes.

The project at Lilly Lake should receive intermittent monitoring to determine if macrophytes will continue to be a problem.

Additional case histories have been described by Peterson (1982a). Most are not as clearly successful as the Lake Trummen project, but dredging to control algal productivity seems more likely to succeed than macrophyte control projects unless depth requirements are better defined and adhered to. The high cost of dredging projects dictates that their objectives are well defined and that plans are well thought out and executed in the best fashion possible.

(8) Costs

The relevance of cost comparisons for sediment removal is questionable because a large number of variables affect the cost figures. These include the type of equipment used, the project size (volume of material to be removed), availability of disposal site, the density of the material being removed, the distance to the disposal area, and the ultimate use of the removed material. Data for 64 lake sediment removal projects in the United States indicated a cost range from \$0.24 m^{-3} to \$14.00 m^{-3} (Peterson, 1981). Given the above variability, an average cost figure is not helpful in estimating costs for a single project. However, hydraulic dredging costs ranging from \$1.25 m^{-3} to \$1.75 m^{-3} are relatively common and probably can be considered "reasonable." If sediment contaminated with toxic substance is encountered and special dredges and/or treatment methods are required, dredging costs may exceed \$25.00 m^{-3} (Matsubara, 1979; Koba et al., 1975; Barnard and Hand, 1978). In general, the per unit volume cost of sediment removal will be inversely related to the total volume of material removed. Any time dredged material is of sufficient quality for use as potting or top soil dressing, as was the case at Lake Trummen, Sweden (Sjön Trummen i Växjö, 1977); Nutting Lake, Massachusetts (Worth, 1981); Lilly Lake, Wisconsin (Dunst et al., 1984); and Paradise Lake, Illinois (Lembke et al., 1982); the overall project costs might be reduced significantly.

With cost comparisons being extremely difficult among sediment removal projects themselves (Peterson, 1981), it may seem impossible to make realistic cost comparisons of sediment removal with any other lake treatment technique. However, Peterson (1982b) attempted to do so by making the basic assumptions that both treatment approaches are aimed at the control of inlake nutrient cycling and that nutrient inactivation and dredging treatments are based on criteria thought necessary to achieve the necessary reduction in nutrient cycling. Thus, treatment costs should reflect the amount of phosphorus removed (dredging) or sealed into the lake system (nutrient inactivation) to prevent excessive internal cycling.

Direct cost comparisons of the two treatments cannot be made since nutrient inactivation costs usually are reported on the basis of materials and labor required to treat a hectare of lake surface (area); while dredging costs are computed on the basis of cost to remove a cubic meter of sediment (volume). Peterson (1982b) used per cubic meter dredging costs, lake area, percent of basin area dredged, and dredge volume to calculate the dredging cost on a per hectare basis. If the assumption is correct that only sufficient sediment material was removed to control the internal nutrient cycling (lake deepening projects per se were not included in the calculation); then dredging costs and nutrient inactivation costs should be comparable. Calculations were made using nutrient inactivation cost data from Cooke and Kennedy (1981) and dredging cost data from Peterson (1981). Results of the calculations are shown in Table 8.6.

The nutrient inactivation costs in Table 8.6 do not include equipment costs, but that expenditure for this type of treatment is generally considered to be small relative to other costs (about 9% of total cost at Medical Lake, Washington). Equipment costs added to the Medical Lake project would increase them by \$256 ha^{-1} bringing the most expensive (per hectare) nutrient inactivation project cost close to the least expensive dredging project costs. Labor costs for nutrient inactivation in Table 8.6 assumed an 8 hour day and \$5.00 per hour. The hourly labor cost may be conservative. However, even if doubled it would not make dredging generally competitive with nutrient inactivation, if cost alone is considered.

To put the costs in Table 8.6 in proper perspective, it should be noted that the Medical Lake project costs are abnormally high. They represent real costs, but are high because the lake has steep banks, is relatively deep ($\bar{z} \approx 10$ m and $z_{max} = 18$ m), has a large volume (6.2×10^6 m^3), and an uncommonly high total alkalinity (approximately 750 mg $liter^{-1}$ as $CaCO_3$). The large

Table 8.6. Per hectare cost comparison of dredging and alum treatment to control in-lake phosphorus dynamics in twenty treated lakes. Costs were calculated from selected data of Peterson (1981) and Cooke and Kennedy (1981).

	Lake	Treatment Type	Lake Physical/ Chemical Data	Chemical and Dose	Sediment Removed (m^3)	Treatment Cost ($/ha)
1)	Horseshoe Lake, Wisconsin	Liquid alum	A_o = 8.9 ha V = 3.6×10^5 m^3 Z_m = 16.7 m $\bar{z}$ = 4.0 m Alk = 218-278 mg/1 pH = 6.8-8.9 dimictic	2.6 g Al/m^3	-	101
2)	Lake San Marcos	Liquid alum	A_o = 18.2 ha V = 4.3×10^5 m^3 Z_m = 2.3 m $\bar{z}$ = 2.3 m Alk = 190-268 mg/1 pH = 7.3-9.1	6.0 g Al/m^3	-	126
3)	Welland Canal Ontario, Canada	Liquid alum, surface	A_o = 74 ha V = 6.2×10^6 m^3 Zm = 9.0 m $\bar{z}$ = 9.0 m	2.5 g Al/m^3	---	204

			Alk = 109 mg/l dimictic			
4)	Mirror Lake, Wisconsin	Luquid alum and aeration	A_o = 5.1 ha V = 4 x 10^5 m^3 Zm = 13.1 m $\bar{z}$ = 7.8 m Alk = 222 mg/l pH = 7.6 monomictic	6.6 g Al/m^3	-	400[1]
5)	Shadow Lake, Wisconsin	Liquid alum	A_o = 17.1 ha V = 9.1 x 10^5 m^3 Zm = 12.4 m $\bar{z}$ = 5.3 m Alk = 188 mg/l pH = 7.4 dimictic	5.7 g Al/m^3	-	400[1]
6)	Cline's Pond Corvallis, Oregon	Liquid sodium aluminate and HCl	A_o = 0.4 ha V = 9600 m^3 Zm = 4.9 m $\bar{z}$ = 2.4 m	10.0 g Al/m^3	-	420

		Alk = 30-50 mg/1 pH = 7.0-7.7 monomictic			
7) West Twin Lake, Ohio	Liquid alum	A_o = 34 ha V = 14.2 x 10^4 m^3 Zm = 11.5 $\bar{z}$ = 4.4 m Alk = 102-149 mg/1 dimictic	26.0 g Al/m^3	-	425
8) Dollar Lake, Ohio	Liquid alum	A_o = 2.2 ha V = 0.86 x 10^5 m^3 Zm = 7.5 m $\bar{z}$ = 3.9 m Alk = 101-127 mg/1 pH = 6.7-8.6 dimictic	20.9 g Al/m^3	-	504
9) Medical Lake, Washington	Liquid alum	A_o = 64 ha V = 6.4 x 10^6 m^3 Zm = 18 m $\bar{z}$ = 10 m	12.2 g Al/m^3	-	1,740[2]

		Alk = 750 mg/l pH = 8.5-9.5			
10) Half Moon, Wisconsin	Dredging, ~ 30% of basin	A_o = 53.4 ha V = 8.9 x 10^5 m^3 Zm = 2.7 m $\bar{z}$ = 1.7 m	-	25 x 10^3	2,137
11) Lilly Lake, Wisconsin	Dredging, 100% of basin	A_o = 35.6 ha V = 5.3 x 10^8 m^3 Zm = 1.8 m $\bar{z}$ = 1.4 m	-	680 x 10^3	4,584
12) Commonwealth, Oregon	Dredging, 100% of basin	A_o = 2.6 ha $\bar{z}$ = 0.9 m	-	19 x 10^3	5,769
13) Steinmetz, New York	Drain and bulldoze, 75% of basin	A_o = 1.2 ha V = 8.1 x 10^5 m^3 Zm = 2.1 m $\bar{z}$ = 1.5 m	-	2 x 10^3	7,233
14) Carnegie, New Jersey	Dredging, 75% of basin	A_o = 110 ha Zm = 3 m	-	765 x 10^3	12,054
15) Lenox, Iowa	Dredging, 100% of basin	A_o = 13.4 ha Zm = 3.4 m	-	76 x 10^3	13,328

			$\bar{z}$ = 0.9 m			
16)	Nutting, Massachusetts	Dredging, 56% of basin	A_o = 31.6 ha Zm = 2.1 m $\bar{z}$ = 1.3 m	-	275×10^3	18,858
17)	Sunshine Springs, Wisconsin	Dredging, 100% of basin	A_o = 0.4 ha V = 1.7×10^3 m^3 Zm = 1.2 m $\bar{z}$ = 0.5 m	-	5.1×10^3	26,832
18)	Krause Springs, Wisconsin	Dredging, 100% of basin	A_o = 0.3 ha V = 0.98×10^3 m^3 Zm = 1.0 m $\bar{z}$ = 0.34 m	-	4.9×10^3	31,754
19)	Collins Park New York	Dredging, 15% of basin	A_o = 24.3 ha V = 6.5×10^5 m^3 Zm = 9.75 m $\bar{z}$ = 2.7 m	-	52×10^3	39,178
20)	59th Street Pond, New York[3]	Drain and bulldoze, 100% of basin	A_o = 1.8 ha $\bar{z}$ = 0.2 m	-	13×10^3	100,605

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[1] Cost per hectare, spread over both lakes as a unit (5.1 ha + 17.1 ha = 22.2 ha).
[2] From Gasperino et al. (1980).
[3] All sediment from 59th Street Pond had to be trucked out of New York City, thus escalating the costs.

volume and high alkalinity required greater amounts of aluminum sulfate to reduce the dissolved reactive phosphorus concentration by the targeted 87 percent. These factors make Medical Lake somewhat unusual among the alum treated lakes. The 59th Street Pond dredging costs also were unusually high for the reasons noted in Table 8.6. If these two outliers are eliminated from Table 8.6 and the median cost per hectare of the remaining projects is computed, it can be seen that nutrient inactivation (\$400 ha^{-1} is quite attractive relative to dredging (\$12,691 ha^{-1}) when cost alone is examined.

It might be argued that dredging and nutrient inactivation costs should not be compared since dredging addresses one set of lake problems while nutrient inactivation addresses another. Peterson (1982b) concluded that was not necessarily the case. He found considerable overlap between the lake conditions suitable to dredging and nutrient inactivation. A possible exception to the similarities was depth and rooted plant growth. There is still some question about the effectiveness of nutrient inactivation in shallow lakes which are generally ideal candidates for dredging. However, the experience of Welch et al. (1982) at Long Lake, Washington adds credence to the idea that nutrient inactivation can be effective in shallow as well as deeper lakes.

The costs in Table 8.6 were calculated as "end of treatment" costs, i.e., what it actually cost to complete the treatment. Costs of this nature are misleading. To be meaningful the treatment costs should be amortized over the effective life expectancy of a project. Unfortunately, lake restoration is new enough that this kind of cost figure cannot be generated with any confidence. Continued monitoring of lake restoration projects will improve this situation. Preliminary evidence indicates that initially successful dredging projects may have greater duration of effectiveness than nutrient inactivation. However, that is not definitive. If it proves, over time, to be the case, then it should become possible to determine if the initial high cost of dredging compared to nutrient inactivation treatments is warranted.

If the amortization of cost rationale is applied to the Lake Trummen (not shown in Table 8.6) dredging costs, they look much more reasonable than the end of project costs. The 1971 cost of the entire project was $572,222 (using a conversion factor of 4.5 Sw. Kr. per dollar). This translates to a cost of approximately $1,430 ha^{-1}. If the cost is amortized over the post treatment years the lake has shown benefits (9 years), the costs translate to approximately $159 ha^{-1} yr^{-1}. This figure will continue to decline as long as the lake maintains its current level of quality. A figure such as the cost ha^{-1} yr^{-1} makes it much easier to compare the "real cost" of treatment across different treatment types since the cost can be calculated on the basis of effectiveness. For example, if the above costs for Lake Trummen dredging are compared to the cost of treating West Twin Lakes, Ohio we see the following. West Twin Lake is 34 ha. The cost to treat it with alum in 1975 was $425.00 ha^{-1}. Cooke et al. (1981) have shown the treatment benefits held for five years beyond treatment (not measured since then), thus the costs would be about $85 ha^{-1}

yr^{-1}. From this brief, rough analysis, nutrient inactivation still holds the edge over dredging in terms of cost. However, a much larger sample of cost comparisons is needed to draw any valid conclusions concerning dredging costs vs. nutrient inactivation costs. Only time (determination of restorative effects) will tell the real story. This type of assessment will apply only to cases where treatment is directed toward limiting internal nutrient cycling. Dredging is still the only practical method of deepening a shoaled lake if deepening per se is the objective, or if littoral zone removal of macrophytes is targeted as in Lake Trummen. Alum treatment will apparently have limited or no effect in reducing macrophyte biomass, and may even enhance plant growth by increasing light penetration.

(9) Summary

There are four major reasons for sediment removal from a lake. They include deepening, limitation of nutrient recycling, reduction of macrophyte nuisances, and the removal of sedimented toxic substances. Environmental concerns associated with lake sediment removal appear to be less negative than might be expected. Case histories reveal that while adverse impacts do occur, they generally are not severe and are of short duration. Another possible long-term impact of dredging is the effect dredged material has on the area where it is disposed.

Dredging depth will depend on the purpose of the project and the sediment characteristics of the lake. Two different approaches to determining dredging depth for the control of in-lake nutrient cycling from the sediments are cited. A third method has been proposed when the intent is to control macrophytes. An important aspect of any sediment removal project is a thorough pretreatment evaluation of nutrient and sediment mass balance. It will help determine the feasibility of dredging and provide insight to the expected longevity of effectiveness.

Almost all projects designed to deepen a lake are successful. Those designed to control internal nutrient cycling have had mixed results. However, most failures can be traced to incomplete pre-evaluation or simply removing too little sediment. Macrophyte control projects have not been evaluated extensively enough to permit generalization. Toxic sediment removal projects have been completed successfully, but generally require use of special dredges and disposal practices. Numerous specialized techniques are available.

Cost comparisons among dredging projects are difficult due to the large number of variables involved. Dredging cost comparisons with other treatment techniques are not commonly made because of the many variables and the non-uniform units of measuring costs, i.e., \$ ha^{-1} of treated area for nutrient inactivation; \$ m^{-3} of sediment dredged. A few basic assumptions permit the conversion of the different costing units into common terms. All of the other variables still exist, however, it appears that dredging is several times more expensive than nutrient inactivation to accomplish the same short-term results (minimization of internal nutrient recycling) if cost alone is considered. "Real cost" determination must be deferred until the longevity of treatment effectiveness for each technique is evaluated and the cost can reasonably be put in terms of $ ha^{-1} yr^{-1}.

There is little doubt that dredging can be a successful lake restoration technique if there is a thorough pre-implementation evaluation of the lake setting, if proper equipment is selected, if the disposal areas are designed for end of treatment effectiveness, and if the dredging is conducted by conscientious and competent operators. Successful examples of both surface sediment skimming and deep (6 to 8 m) dredging can be cited. Thorough pre-implementation evaluatin is the most important aspect of the project.

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